BEHAVIOUR OF POST TENSIONED SEGMENTAL CONTINUOUS BEAM UNDER PULSATING LOADS

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By

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CERTIFICATE

This is to certify that the thesis entitled 'BEHAVIOUR OF POST-TENSIONED SEGMENTAL CONTINUOUS BEAM UNDER PULSATING LOADS' submitted by K. CHANDRASHEKHAR IYER for the award of M.Tech. degree in Structural Engineering is a record of work carried out under my supervision and has not been submitted elsewhere for a degree.

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DEDICATED

TO THE

MEMORY OF

MY FATHER AND SISTER

ACKNOWLEDGEMENT

I feel immense pleasure in expressing my sincere thanks to Dr. P. Dayaratnam, Professor of Civil Engineering who inspired me to undertake a thesis work on 'Prestressed Concrete'. The present work is the result of incessant inspiration I got from him and his able guidance.

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K.C. IYER

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ABSTRACT

Post-tensioned segmental construction is one of the widely used techniques of prestressed concrete construction. The main aim of the investigation was to study the influence of number of segments on the behaviour and the ultimate load capacity under static and pulsating loads.

Two sets of two-span continuous beams were tested. One set of beams was tested under static loads with two point loads placed symmetrically with respect to central support. The deformation and ultimate load under static condition were measured. Another set of beams was tested under pulsating load. The deformations ultimate loads and number of cycles resisted by the beams were measured. The upper and lower loads were decided on the basis of ultimate capacities found from the static tests.

The ultimate load carried by the beams of the first set checked well with the theoretically computed values which were computed without any moment redistribution factors. Single unit and five-segment beams indicated full redistribution while the seven-segment one resisted 86 percent of the theoretical value. Beams under pulsating loads survived for only few hundred thousand cycles with upper load in the range of 50 to 60 percent of static ultimate load and lower load about 33 percent of ultimate load. Full moment

redistribution under pulsating loads did not take place because of continuously increasing deformation and limiting strain in concrete.

One should anticipate larger deflections in segmental construction therefore adequate design provision be made. Properly bonded non-tensioned reinforcement may be advised for longer survival of the beam under pulsating loads. The shear keys resisted the shear free adequately, however the compression fibres at the joints are the weak links and failure of secondary compression occured at the joints.

CHAPTER - 1

INTRODUCTION

1.1 General:

Prestressed members are frequently employed in various types of constructions such as bridges, industrial and apartment building systems, trusses, electric poles, railway sleepers, cylindrical shells, water retaining structures etc. In some cases of long span bridges, where mainly self weight and constructional methods govern the cost, prestressed concrete construction is found to be more economical than ordinary reinforced concrete construction.

In the various types of construction techniques used for prestressed concrete construction one of the most promising techniques is the segmental construction. In the segmental construction, large quantities of high quality precast units (or segments) are manufactured under controlled factory conditions and then transported with conventional carriers to the job site. There, the segments are assembled easily and lifted into a place on the superstructures. The segments themselves are tied together using post-tensioning.

1.2 Advantages:

The precast segmental construction has following advantages:

i) Control of high quality and precise dimensional tolerances.

ii) Curing cost is small. Steam curing is also possible for high early strength. Curing in moist atmosphere can be carried out for all 28-days.

iii) Construction is not greatly influenced by weather conditions. The precast segments can be mass-manufactured and brought to site at any desired time.

iv) Loss of prestress due to shrinkage and creep is reduced, as the prestressing is done after it has gained full design strength.

v) False work and temporary supports are usually minimized as well as related hazards due to unknown soil conditions. If needed, segments have to be supported only till prestressing is done instead of entire span support.

vi) The speed of erection may be chosen to suit the job requirements. Segmental construction is very suitable in transportation structures and is a fast construction method (1).

vii) It causes minimum traffic disruption in congested urban areas. Where clearance requirements during construction are critical, there may be further savings on the approach structures because no head room is needed for the form work.

viii) Over all labour requirement is less than that for conventional methods while a major part of the work on site is replaced by plant labour (1).

ix) The technique shows an exceptionally high safety record (1).

x) If required,very easy to dismantle during war time.

xi) By a small variation in precasting operation (or of jointing material between them), a structure can be produced to take up any required horizontal or vertical curvature as well as the required super-elevation.

xii) Because of staged construction, the curtailment of cables and provision of draped cables are possible.

xiii) Long term deflections,which are very much dependent on the age of concrete,are significantly reduced as precast segments are usually stored for few weeks before construction.

xiv) In case of medium to long span structures,it has been shown to be an efficient and economical constrution. Jean Muller (1)claims that cost savings in comparison to conventional construction methods have reached 20 percent on several projects in U.S.A.

A major constraint or disadvantage is that the operation or the construction technique requires an excellent transportation and handling facility. This is not easily available in all

places in India. Irrespective of advantages or disadvantages the construction method has its role to play in some projects.

1.3 Continuous Beams:

In an efficiently designed continuous beam, it is possible to have an appreciable saving of material, both concrete and steel, as compared to equivalent simple spans. Continuous beams are stiffer than simple beams, so it is possible to use a smaller depth for a continuous beam without decreasing its stiffness. Freyermuth (2) claims that the use of continuity permits a reduction of 5 to 15 percent in required prestress force when compared to single span. The efficiency of design depends not only on the proportioning of concrete section and prestressing force but also on the number of spans and ratio of span lengths. It has been observed that greater reduction in prestressing force occurs in shorter span bridges where the live load plus impact moments are a larger portion of the total design moment.

Continuous beams have the advantage to be designed to have more ductility and strength than is possible in equivalent simple beams.

Primary reason of using continuity with precast prestressed girders is the elimination of maintenance cost associated with bridge deck joints and deck drainage onto substructure.

Continuity also improves the appearance and the riding quality than simple span girders on the piers.

1.4 Methods of Manufacturing the Precast-Segments:

There are basically two methods in use for the manufacture of precast segments.

(i) Casting Beds Method: This method is more commonly known as 'Long-Line Method'. In this method an entire or one half the span of cantilever or beam is cast on a bed which reproduces exactly the same profile of the deck soffit with due allowance for camber. When many sets of form work are available, several different segments may be cast at the same time on a long line.

In this method the position of each segment is fixed and the form-work moves along the bed. The main advantage of this method is that it is easy to set up and maintain control over the production of the segments. After stripping the forms, it is not necessary to take away the segments immediately. It has got a serious disadvantage that it requires a large space. The minimum length is normally slightly more than half the length of the largest span of the structure.

(ii) Casting Machine Method: This method is commonly known as 'Short-Line Method'. In this method, single casting units are designed to have a variable geometry corresponding to

bridge profile. Here the form work remains fixed while the segments progress from the casting position to the match-cast position. The mould soffit remains with each segment until removed for storage.

This soffit is equipped for longitudinal transfer and position adjustment.

The external forms are usually hinged for easy stripping. The internal form is of collapsible type with removable lower panels for height variation as required.

Vertical castings of the segments (i.e. cast on edges) can also be employed for easier concrete placing and vibration.

The main advantage of this method is that the space needed is small, approximately three times the length of a segment. The horizontal and vertical curves and twisting of the structure is obtained by adjusting the position of the neighbouring segment where as in long line method the long-line has to be designed to accomodate the curvature.

The main disadvantage of this method is that the concrete at the top of the segments which are intended for horizontal use, but which are cast vertically may be of lower quality and has to be finished properly.

1.5 Methods of Erection:

Several methods have been developed for placing the precast segments on the super-structure. They are classified as follow:

(i) Cranes: Mobile cranes moving on land or floating on pantoons are commonly used where access is available.

(ii) Winch and Beam: In this method, a lifting device attached to an already completed part of the deck raises the segments which have been brought to the bridge site by the land carrier or barge. The segments are lifted into place by winches carried at the deck level on a short cantilever mechanism anchored to the bridge.

(iii) Launching Gantry: Here a special mechanism travels along with the completed spans and maintains the work flow at the deck level.

In all the above methods the final section completing the bridge is either cast-in-situ to the actual dimension or is precast exactly to the required dimensions.

1.6 General Consideration on Precast Segmental Construction:

1.6.1 Concrete:

Uniform quality of concrete is essential for segmental construction. PCI (13) recommends that ideal concrete for segmental construction will have as near as practical zero slump. Proper vibration should be used to afford use of lowest

slump concrete and to allow for the optimum consolidation of concrete. 28 - day strength according to PCI must be greater than the strength specified by structural design and here minimum strength is not specified. AASHTO (17) specifies that 28-day strength shall be not less than 35 N/mm^2.

1.6.2 Joints:

Joints are the weakest points of the segmental construction. Joints which are nearer to the critical section are more vulnerable as failure due to combined action of stresses are predominant. Monolithic behaviour of beams is mainly dependent on the joints and the joint surfaces. Best suited joints for prestressed segmental construction are described below:

(a) Cast-in-place joints: Here the segments are kept at 100 to 150 mm apart and duct is made continuous. Then using the form work the gap is filled with relatively rich mix using high early strength cement.

PCI (13) recommendation regarding the design width of the joint : it must allow access for coupling of conduits, welding of reinforcement and thorough vibration of concrete. Typical joint widths are twice the thickness of the web or half the flange width but not less than 100 mm.

Such joints are not good for shear transfer.

(b) Dry Joints: In this method the joints are placed directly against each other without placing any material in between. The construction requires careful techniques in manufacturing and matching the individual joints. By this method of jointing the speed of construction and economy increase considerably. But for perfect match of the joint surfaces,surfacing or grinding becomes essential some times which again increases the cost of construction. But careful construction in the first stage itself eliminates this expense.

(c) Mortar Joint or Grouted Joint: This type of joint is also called as 'Buttered Joint'. The joint can be formed by filling the gap between the segments with mortar. The width of joint ranges from 30 mm to 50 mm. The joints may be made with either gravity or pressure method. For purpose of gravity method grout or mortar consists of approximately one part of cement, one-third part of clean fine sand and water reducing agent. In pressure method,the joint must be tightly sealed in order to sustain pressure.

These joints are not good because of concentration of stress due to inequality of mortar or grout.

(d) Epoxy-bonded joints: Here 0.8 mm thick epoxy resin is generally used for joining the segments. There should be perfect fit between the two adjacent segments. These joints give high compressive strength and the tensile strength

equal to the tensile strength of concrete. This type of joint is widely used now-a-days, because it produces very nearly to a monolithic structure.

(c) Metal Joints: Metal joints in the form of steel plates which are welded together or pin connected metal hinges, etc. may be used. Special attention has to be provided for welded joints to prevent cracking or spalling of the concrete and to prevent straining of adjacent surfaces.

1.6.3 Jointing Surfaces:

For match-cast joints, the surface including formed keys should be even and smooth to avoid point contact and surface crushing or chipping off of edges during post-tensioning.

For wide joints the rough surfaces are preferable as they produce better bond between segments and filling material. The adjacent concrete surface should be kept thoroughly wet for approximately 6 hours or a bonding agent should be applied prior to the construction of joint (13).

Jointing surfaces should be oriented perpendicular to the main post-tensioning tendons to minimise shearing forces and dislocation in the plane of the joint during post-tensioning. According to PCI, inclination with respect to a plane perpendicular to the longitudinal axis is permitted for joints with assured frictional resistance. The inclination should generally not exceed 20 degrees. Larger inclination,

but not more than approximately 30 degrees, may be permitted if inclined surface area is located close to neutral axis and does not exceed 25 percent of the total joint -surface area.

1.6.4 Keys:

Purpose of keys is to transfer shear and facilitate the alignment of segments during erection. It is recommended by PCI that keys be proportioned for erection loads so that the bearing stress and shear stress on the gross sectional area of key do not exceed the allowable values. They may be male-female type or castellated type.

1.6.5 Grouting:

Grouting is done to achieve bond between steel and sheath or duct. Expected behaviour of grouted beam is same as pretensioned beam, except near the anchorage.

1.6.6 Selection of Transverse Cross-Section:

In general, box girders have been shown to be the best suited for most design and construction requirements. In addition, the torsional rigidity of the section provides excellent stability during construction and later during the life of the structure.

Jean Muller (1) suggests that depending upon the deck width, the design of the transverse cross-section will vary. For example,

i) upto 12 m , a conventional single box with two webs may be used ;

ii) for wider decks, several individual boxes are assembled transversely by prestressing. However, for single box girders, a more refined design should be used like three or four webs (with vertical or tapered faces) for widths upto 21 m;

iii) for intermediate widths, single box girders may be used in conjunction with a ribbed roadway slab or boxed cantilever.

According to Sinha (10) ,in a single box girder design , the spacing of webs is determined by the rigorous structural analysis which can be reduced if span to depth ratio is kept nearly 30 at which it behaves like ordinary beam. If the top flange width is nearly 7.5 percent of the span, observation suggests that cantilever part ,'c' (Fig. 1.1) of the flange in cross-section can be kept 1/4 of b_f. The dimensions b, c, b_{f1}, b_{f2} are decided by transverse moment criteria. Thickness of web, b_w is based on minimum thickness for anchorage, hardware and capacity to resist bursting force. Check for principal stress is provided if the shear stress and bonding stress govern the design. Sometimes the prestressing and partial longitudinal prestressing is done to take care of the handling stresses.

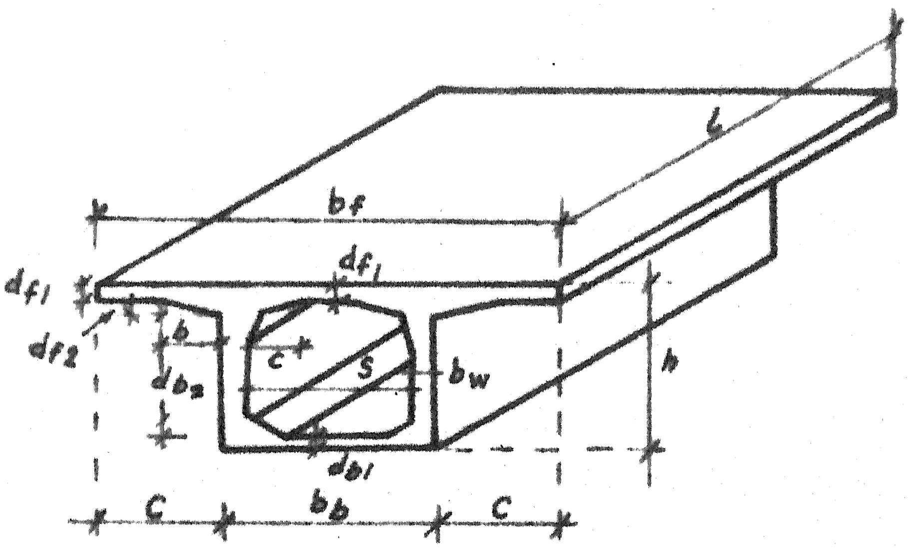


Fig 1.1 TYPICAL BOX BEAM SEGMENT

In general mild steel reinforcement is provided for the purpose of monolithic action of webs and flanges, i.e. to take care of transverse BM to sustain the handling stresses and to resist torsion.

1.6.7 Selection of Longitudinal Section:

A constant depth design has the advantage of simplicity for construction and is sometimes preferred for aesthetic reasons. It has been observed that constant depth is not economical for the spans much longer than 60 m in the segmental construction.

For longer spans, a variable depth with circular soffits or straight haunches are also used.

The length of the segment governs the number of segments in the span, cost of segments, cost of transportation and erection. The weight and size limitation usually determine the value of segment length.

1.7 Literature Review:

A brief review of literature on sustained as well as repeated loading effects on prestressed concrete structure is presented.

The effect of pulsating load on beam is the development of fatigue to certain extent and incremental collapse. Fatigue causes fracture as a result of repeated loading, If the deflection during each cycle goes on increasing, the cumulative

deflection may cause collapse and it is called 'incremental collapse'. If cumulative deflection becomes constant after few cycles of load the structure is said to have reached a 'shake down limit'.

Before investigating the behaviour of beam under pulsating load, it would be essential to study the behaviour of individual element of the beam by which it is made up of, viz., prestressing steel, mild steel and concrete.

1.7.1 <u>Behaviour of Steel and Concrete Under Pulsating Loads</u>:

At one time the endurance limit based on one million repetitions of load was considered to be sufficient, but these days two or more million repetitions are frequently applied to any material.

Ekberg, Walther and Sluther (3) represented behaviour of prestressing steel by 'Goodman' diagram which is a graphical form of the results obtained from tests on axle steel for one million cycles of repeated loading. They concluded that if the lower stress is zero and only tensile stress is applied, the range of stress is usually about 30 percent of stress at which static failure occurs for the failure to cause after one million cycles.

Abeles and Bardhan Roy (4) have concluded that the endurance limit of cold drawn wire for two million repetitions lies between 55 percent and 70 percent of the ultimate strength

when the range of dynamic stress lies between zero and endurance limit. They further concluded that the endurance limit may lie between 66 percent and 75 percent of ultimate load for a much smaller range of dynamic stress.

Abeles on the basis of tests conducted by Ros,M. and Sarrasin, A. in Switzerland showed the reduction in the strength of concrete prism subjected to two million cycles of repeated compressive loading and it depends upon the mix used. He also showed that endurance limit reduces when the range of dynamic loading increases. According to 'Goodman'diagram for concrete it is minimum for stress range of zero and 66 percent of prism strength.

S.P. Shaw and S. Chandra (5) studied the mechanics of failure of concrete subjected to slowly applied cyclic or sustained stresses. They observed the crack growth in two stages. Stage I crack growth for specimens which did not fail within the test period and Stage II crack growth with failure. They concluded that Stage I crack growth occurred only in long time loading and Stage II crack growth occurred primarily due to the composite nature of concrete. They also studied the growth of microcrack with the help of ultrasonic and microscopic measurement and concluded that sustained loading has less damaging effect than corresponding cyclic loading for non-failed specimens.

1.7.2 Prestressed Concrete Beam under Repeated Load .

Many works have been done in this field and most of them have considered the simply supported beams.

Abeles (16) concluded that prestressed concrete is not usually subjected to a large range of dynamic loading except in the case of special constructions such as railway sleepers, foundation for turbines and small span bridges. In normal structures in which dead load is considerable, the range is limited to that portion of live load which recurs regularly. He recommends that occasional loads have little long term effect if they occur at intervals of thirty minutes or longer, and may be considered as static loads. It is usually sufficient to ensure that the factor of safety against failure under static loading is obtained after one or more millions of cycles of the range of dynamic loading have been applied.

Sawks (6) and Price (7) inferred that flexural stiffness decreases and permanent set increases with increased repetition of loads. Flexural tensile cracks develop towards the top of the beam with increase in repetition of loads. At the low load levels crack propagation terminates after several million load cycles. Sawks also suggested restressing of tendons to increase flexural stiffness.

Chung (8) investigated the response of unbonded prestressed concrete beam to repeated loading and subsequent

behaviour of beams. The variables were the prestressing steel and maximum level of repeated load. Results obtained indicated that ultimate strength is not significantly affected by repeated load, provided that the beam does not fail in fatigue, but the deflection and crack width increases substantially under repeated loading. He also noticed that flexural rigidity had increased by repeated loading if the maximum load level was below 30 percent, but beyond that the trend was reversed. When maximum load level was in the range of 67 to 75 percent of ultimate capacity, the beams failed under repeated load due to fatigue fracture of steel at the anchorage or near the mid span.

Abeles and Turner (16) have the same feeling that appearance of hair cracks has no effect on the resistance to fatigue, so long as the crack remains fine. The resistance to fatigue is not reduced even after many million cycles of loading. As soon as cracks become wider, however, the bond is destroyed near the cracks and the resistance to fatigue then depends only on the magnitude of the prestress and properties of the steel which cause 'notch effect'. They further concluded that the effect of dynamic loading is much less severe in structures with well bonded steel than in structures with unbonded or poorly bonded steel.

Bobrowski (14) worked on resistance to impact and found that bonded tendons should be properly anchored, otherwise

they get displaced by sudden impact and leave the member like a missile. He also observed that the prestressed members are thinner and shallower which brings the natural frequency of member close to the frequency of normally applied load causing nearly resonance condition.

Patnaik (9) studied the behaviour of prestressed concrete beams subjected to variable pulsating load. He showed that there is no danger to ultimate capacity of the beam even if it is subjected to occassional loading of 75 percent of its ultimate capacity keeping the normal load in the range of 60 percent of its ultimate strength. Wide cracks were observed in beams at peak loads which reduce considerably at normal loads and disappear at lower loads.

Sinha (10) carried out the experiment on box sectioned segmental prestressed beam under pulsating load at I.I.T.Kanpur. He observed that beam with larger eccentricities have higher flexural stiffness along with higher moment capacity than that of smaller eccentricities. However, the advantage of higher stiffness was lost under pulsating loads as cumulative deflection in beams having larger eccentricities was larger when compared to the smaller eccentricities. He also recommended that unbonded box section should be designed for 20 percent extra deflection to sustain repetitive loads.

Sinha (10) studied the effect of epoxy joint over cement slurry and observed that cement slurry joints caused

large and unstable vibration of beams and beams with epoxy joints are even better than monolithic beam with regard to cumulative deflection, creep recovery, residual deflection, ductility and vibration.

Y. Guyon (11) tested four continuous prestressed beams of rectangular section. The beams differed from one another in arrangement of tendon profiles which in some cases had intentionally been given an unsuitable shape. They were loaded by means of two loads placed side by side, at the centre of each span. He observed the formation of plastic hinge before the failure occurred.

G. Macchi (11) investigated three span continuous beams (100 mm x 250 mm, rectangular cross-section). The beams differed in spans and tendon profiles. For purpose of comparison he had constructed the simple beams also. Loading in each case consisted of concentrated load at the centre span. He observed certain amount of moment redistribution in all beams.

T.Y. Lin (12) carried out dynamic and static testing on four two-span beams. He observed that ;

i) under the action of repeated loads, the mode of failure may be entirely different,

ii) the plastic theory may not be applicable at all particularly over reinforced sections,

iii) beam behaved according to the classical elastic theory as closely as could be measured within the working load

which was approximately 0.41 times the ultimate load. In this range load deflection curves were also linear,

iv) between working and cracking loads,beams behaved elastically so far reactions and moments were concerned but load-deflection curves were non-linear. But under repeated loads reactions and moments deviated from values calculated by elastic theory by about 3 to 5 percent. But prediction of cracking using elastic theory was correct,

v) due to plastic action after cracking the appearance of cracks under second hinge might be slightly sooner than claculated.

In Britain, P.B. Morice and H.E. Lewis (11) have studied the problem of moment redistribution. They tested 28 rectangular cross-section beams. Beams differed mainly in tendon profiles adopted. It was observed that almost complete equalization of bending moments had been attained by the time failure occurred.

1.8 Purpose of the Present Work:

The object of the present work is to investigate the behaviour of the post-tensioned, unbonded, segmental continuous beams under static and pulsating loads.

Investigations have already been carried out on the various types of post-tensioned simple beams under various

load conditions and load combinations. Investigations have also been carried out to test the post-tensioned (bonded) continuous beams of single unit under static and repeated loading. But there is a lack of information regarding the behaviour of the post-tensioned segmental continuous beam under pulsating load. Theoretically, one can not superpose the behaviour of segmental simple beam over the behaviour of single unit continuous beam to get the required results.

So the present work is to study how the behaviour of a single unit beam differs from that of segmental beam .

The non-tensioned reinforcement which are provided in the segments for handling stresses are not continuous through the joints. So joints are the weak sections.

Under the load conditions the joint opens as soon as tensile stress is produced in the section. Hence preassigned cracks have been formed in the segmental construction. Now the idea is to see;

i) whether cracks form at the critical locations already predicted ?

ii) is ultimate strength a function of crack formation?

iii) study the formation of plastic hinge and see whether full moment redistribution is taking place !

iv) unlike bonded construction the unbonded construction has much fewer cracks (openings) and in course of loading the cracks propagate in the compression zone

before other plastic hinge is fully formed at another critical location. Hence strain in the concrete becomes large resulting to collapse. Now the aim is to study how exactly this phenomenon takes place and how it varies if number of segments are increased ?

v) how much reduction in the ultimate strength of beam as compared to calculated strength of bonded construction as unbondedness results to average strain and hence reduction in the net tensile stress in steel in case of under reinforced section,

vi) **the number of** cycles the beam with stands before collapse under different amplitudes (P_{max}-P_{min}) and different frequencies.

CHAPTER - 2

EXPERIMENTAL PROGRAMME

2.1 General:

It is stated in the Chapter 1 that the purpose of the experimental investigation was to study the behaviour of prestressed concrete continuous beam under static as well as pulsating loads. For carrying out the experiment 10 beams (specimens), all identical rectangular sections, were cast. The difference in the geometry of the beam was the difference in the number of segments in different sets of beams. The entire experimental programme has been divided into two main subprogrammes.

Test subprogramme 1 consists of tests conducted on the materials of construction for the purpose of design and selection of test specimen, making of mould, casting specimens and its prestressing for the test to be carried out under test subprogramme 2.

Test subprogramme 2 consists of experimental set up, placing the beam onto the test frame, choice of the loads with its frequency and the actual test conducted by loading the specimens.

The test subprogramme 2 had been a repetitive process.

2.2 Test of Materials:

Concrete : For concrete mix puzzolana Portland cement, Kalpi sand and granite metal aggregate were used.

Cement : To assess the quality of cement, different tests were conducted. But the most important of them was 7-day and 28-day compression test on the 75 mm mortar cubes for designing the mix. In the entire subprogramme 1 , two batches of cement were used to cast the total number of beams. Beam numbers 1B1S to 6B5S with an exception of beam number 2B1S were cast of cement of batch number 1 and other beams were cast of cement of batch number 2. The test results of cement are given below:

Cement of first batch

7-day strength = 16.7 N/mm^2

33-day strength = 19.5 N/mm^2

Cement of second batch

7-day strength = 18.0 N/mm^2

28-day strength = 30.7 N/mm^2.

Sand : Sand grading was found as per requirement of IS 2116-1965. The results are

i) Fineness modulus of sand = 2.7

ii) Specific gravity of sand = 2.76

iii) Bulk density of sand = 17.65 kN/m^3.

Aggregate : Aggregate of two sizes 12 mm nominal and 20 mm nominal were used in the mix. Blending of 35 percent of 12 mm nominal size aggregate by weight with 65 percent of 20 mm

nominal size aggregate gave the highest bulk density and the same proportion was used in the mix also. The results of these two aggregates and the blended aggregate are given below:

20 mm size aggregate

Bulk density = 15.60 kN/m^3

Specific gravity = 2.83

12 mm size aggregate

Bulk density = 15.30 kN/m^3

Specific gravity = 2.83

Blended aggregate

Bulk density = 16.30 kN/m^3.

<u>Concrete</u>: Three methods were used in designing the mix M35, and they are (a) Fineness Modulus Method, (b) ACI Method, (c) Trial Mix Method.

Fineness Modulus Method gave the proportion 1:1.88:1.26 with water-cement ratio = 0.35. ACI method gave the proportion as 1:1.15:1.71 with water-cement ratio = 0.35. Trial mix based on previous data available gave the proportion as 1:1:1.85 with water-cement ratio = 0.38. These designs were done for the cement of first batch. Here we notice that F.M. method gave lot of sand in the mix in which water-cement ratio was 0.35 which seems to be very low and it was increased, which thereby decreased the strength of concrete. So this design was discarded. ACI method gave 7-day cube strength = 19.9 N/mm^2 and 28-day cube strength = 29.0 N/mm^2. This mix did not seem

to be workable because of less water-cement ratio, so trial mix of proportion 1:1:1.85 with water-cement ratio = 0.38 was adopted and cube strength gave the values of 7-day strength between 17.4 N/mm^2 to 19.8 N/mm^2 and 28-day strength 28.8 N/mm^2 to 34.1 N/mm^2. IRC,for the quantity of cement in the concrete mix for prestressed concrete bridge construction specifies that cement content shall be not less than 3.60 kN/m^3 of concrete nor shall it be more than 5.40 kN per cubic metre of concrete. ISI specification for the maximum quantity of cement that can be used for any concrete construction is 6.0 kN per cubic metre of concrete. Evidently the above mix adopted shows the quantity of cement to be 6.23 kN per cubic metre of concrete. Since the quality of cement did not conform to the ISI specification , it had to be added in large quantity for higher strength and it never gave any problem of shrinkage, etc. Second batch of cement was used for beam numbers 2B1S, 7B7S and onwards. 2B1S and 7B7S were cast with the same proportion which gave very high strength and that was not needed, so the mix adopted for subsequent beams were 1:1.33:2.47 with water-cement ratio of 0.38.

<u>M.S. Bar :</u> Uniaxial tension test on 3 specimens made out of 6 mm dia bars were conducted on INSTRON-1195. The load - deflection curve is shown in Fig. 2.2.

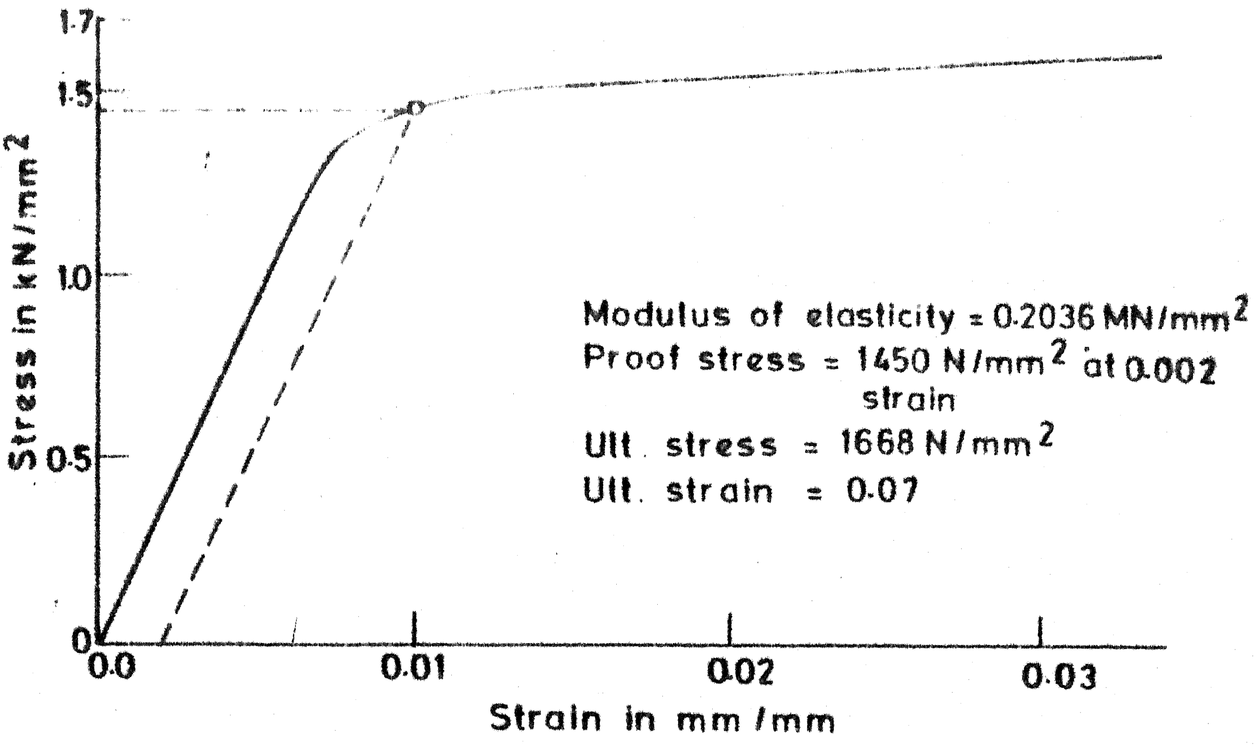


FIG. 2.1 STRESS-STRAIN CURVE FOR 5 mm ϕ H.T. WIRE

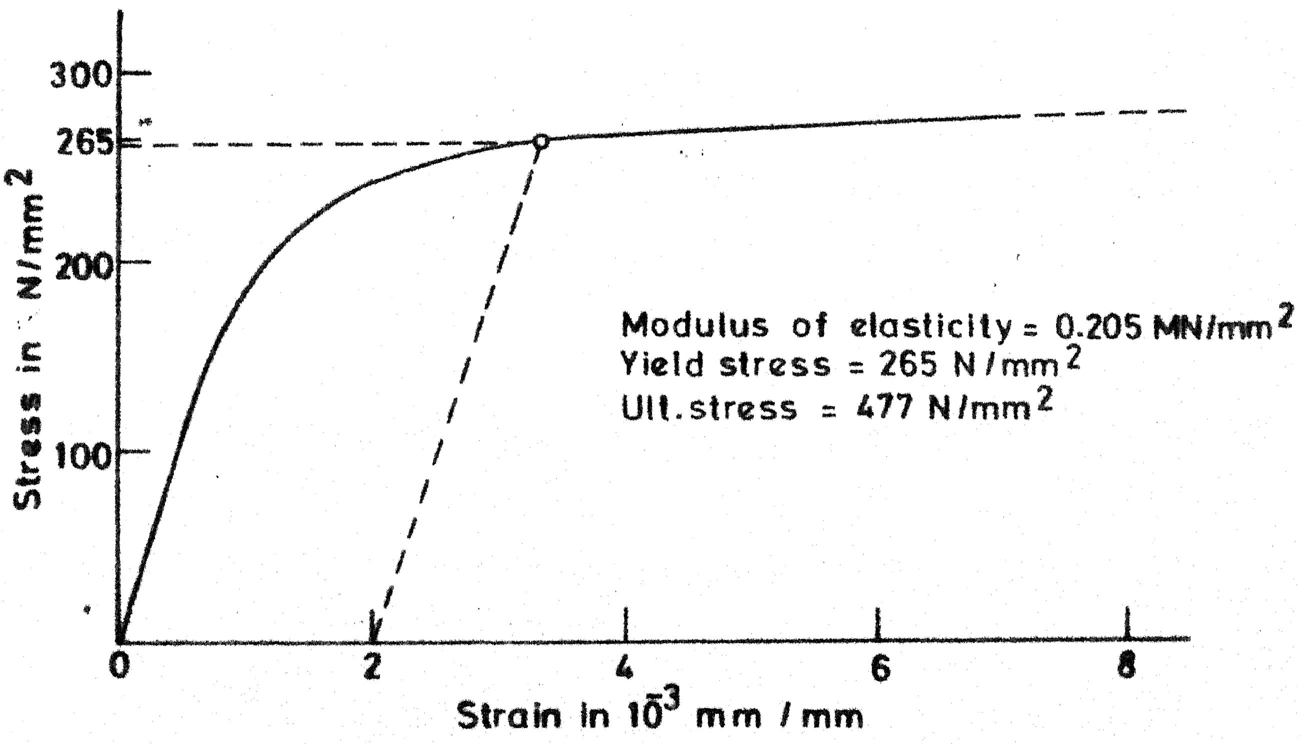


FIG. 2.2 STRESS-STRAIN CURVE OF 6 mm ϕ M.S. BAR

<u>H.T. Wire</u>: Three 5 mm dia H.T. wire were tested for uniaxial tensile strength on M.T.S. machine. Electronic extensometer was also attached to the test specimen and it was connected to the M.T.S. for getting the load versus strain curves. The curves are shown in Fig. 2.1.

Three 5 mm dia H.T. wire were subjected to repeated loads between 50 percent of ultimate load as lower load and 70 percent of ultimate load as upper load at a frequency of 25 hertz. The wires went on for about 37200 cycles before it failed near the grip. PCI specifications (15) for tendons on dynamic tests are that tendons should withstand 500000 cycles from 60 to 66 percent of the minimum specified ultimate strength, and 50 cycles from 40 percent to 80 percent of its minimum specified ultimate strength. The period of cycles should involve the change from lower stress level to upper stress level and return to lower. So, the test wires can be said to have given the desired result to be used for prestressed concrete construction.

TABLE 2.1 : PROPERTIES OF STEEL

S.No.	Properties	5 mm Ø H.T.Wire	6 mm Ø M.S.Bars
1.	Ultimate tensile strength	1668 N/mm^2	477 N/mm^2
2.	Yield stress=proof stress at 0.2 percent strain	1450 N/mm^2	265 N/mm^2
3	Modulus of Elasticity	0.2036 MN/mm^2	0.205 MN/mm^2

2.3 Selection of Test Specimen:

The experimental programme consists of testing of two span continuous beam with each span of 3.0 m length. The span of the beam was assumed before hand and section of the beam was designed on ultimate load theory basis for the defined load point depending on the jacks available. The section adopted was 170 mm x 270 mm solid rectangular section. The beam was cast of uniform section throughout its length. For bearing and other reasons, overhang of 150 mm was provided on either end of the beam.

Some of the reasons clarifying the selection of particular section of the specimen are given below.

Entire sub-programme 2 consisted of testing of 10 beams and it was not possible to cast all the beams right under the loading frame. If it could be done so,the handling of the specimen would have been avoided. But the beams were cast away from the test frame and inside the yard of the laboratory. After 28 days of full moist curing they were brought to the laboratory for prestressing. The specimen were then placed on the loading frame one by one.

In this case full length of 6.30 m beam had to be handled before it was finally brought to the frame. The beam could develop a moment, large enough due to greater lever arm

for the reactions and also impact moment, if it dropped by chance, to damage itself. So small beams had to be preferred to avoid any damage due to handling. On the other hand the span of the beam should be feasible enough to predict for a new structure on the basis of results obtained from it.

Another reason for restricting the length of beam was the capacity of crane, trolley and men associated with it.

Why only rectangular section was adopted for test ? The basic aim of experiment was to study the flexural behaviour of the beams with respect to number of segments. I -section and box-sections are preferred for the construction of pre-stressed concrete beams.

Since the beam was a continuous beam, so it must possess enough moment capacity to resist the negative bending moment at the support. I - section beams are suitable in case of simply supported beams. In case of continuous beams the sections required to be altered from mid-span towards support raising the cost of shuttering (or mould) and hence not considered.

Box sections could be an ideal section, but before going to study the behaviour of box section segmental continuous beam, it was necessary to know the behaviour of a fundamental and general section of segmental continuous prestressed beam which would give a solid base for the development of the

idea for box-section. More over, box sections are preferred where chances of warping are large. In the present experimentation warping was assumed to be negligible.

Since the behaviour of precast segmental continuous beam was being studied for the first time, so a rectangular section was chosen to be the most primary, fundamental and general section.

Two 5 mm H.T. wires were provided both at top and bottom of the section in each beam. The longitudinal profile of the wires had been kept straight. To increase the negative B.M. capacity, a third wire at the top could have been added but minimum spacing between wires required was 60 mm which was governed by the size of needle vibrator (50 mm dia) being used for compaction of concrete. With the spacing of 60 mm (50 mm + 10 mm, as clearance between H.T. wire and vibrator for handling), use of three wires at the top yielded a large section resulting to large weight of the beam and thereby increasing the moment capacity. The beams had to be tested with the available jacks with limited capacity and the beam with three wires at the top could not have been tested with this.

Curtailment of tendons at certain distance from the support was not possible because of two difficulties which might have been encountered, viz. (i) matching of two adjacent segments where one segment is prestressed and another is not,

(ii) filling the gap. So straight tendons extending to full length of the beam were preferred.

The section has also been provided with nominal M.S. reinforcement of 6 mm diameter both at top and bottom. The M.S. bars were provided as secondary reinforcement against handling stresses, thermal stresses and shrinkage.

As already shown, the specimen had been made up of precast segments. The precast segments had to be joined by post tensioning at the required superstructure level to act monolithic. Had the beam not been segmental, pre-tensioning could have been adopted for this length, but the purpose of testing the segmental beam is lost. So pre-tensioning could not be adopted.

More over, in actual structures the H.T. wires are bonded to prevent it from rusting. Bonding is generally done by grouting the duct provided for the tendon. Bonding of the tendons improves the strength of the beam both under static as well as pulsating loads as compared to unbonded construction. PCI recommends (15) that the dynamic tests are not required on bonded tendons unless the anchorage is located in such a manner that repeated load applications are expected on the anchorage. But in unbonded construction, because of occurrence of average strain throughout the length of the tendon, the specimens were tested under repeated loads.

Grouting the duct with the laboratory facilities available required larger duct size which ultimately increased the size of the section and thereby increasing the moment capacity. Here increase in moment capacity was not desired.

The last but not the least problem with grouting is that the specimen had to be left for few days after grouting for grout to gain sufficient strength in itself and across the joint, before it is tested.

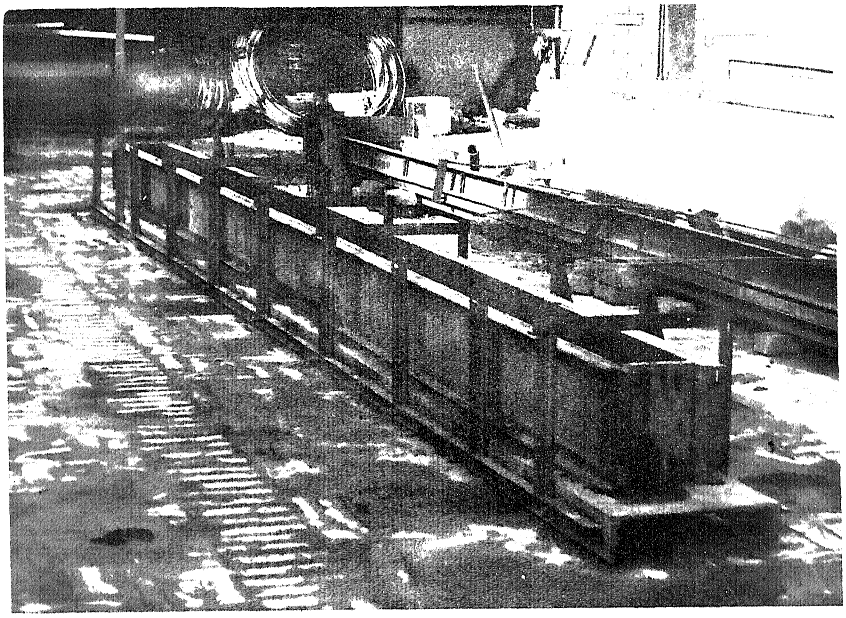
Hence with the limited laboratory facilities and time available, the section adopted seems to be ideal and realistic.

Shear Reinforcement: A number of 2 legged 6 mm Ø M.S. reinforcement were provided at a distance not greater than 180 mm c/c. The actual distance between the stirrups depended on the length of segment.

Shear Key : To transfer the shear between the joints male-female type of joint was designed. The shear key has been shown in plate no. 2.1.

2.4 Mould:

The mould has been shown in plate no. 2.2. The mould used for casting of beams was made up with iron channel (ISMC 400) for base and wooden plank for sides and faces. Iron channel base was preferred more than anything because it would



Mould for Casting the Beam

Arrangement of Segments before Prestressing

give a plane surface at the base which was very essential to be well placed on the loading frame. The channel rested on the firm ground with its flanges supporting it. Two plyboard planks 250 mm thick were used for side shutterings. The sides were 6.5 m long and had to be lifted to dismantle the mould and the plyboards are very weak particularly at the joints where planks meet inside, so these were stiffened with iron angles, ISA 35356 longitudinally both at top and bottom as well as vertically at a distance of 750 mm. The faces of the mould were wooden planks of 25 mm thickness of deodar wood. Faces were firmly attached to one of the sides at both ends. Other side-shuttering was assembled to the faces with the help of screws which could be easily unscrewed while dismantling the mould.

Sides were tied with ties across it at regular interval of 750 mm c/c to prevent it from bulging during vibration of concrete into it. The device was also developed to press the sides and faces downwards so that every point touches the mould base to prevent the flow or leakage of cement slurry from concrete during vibration.

3 mm thick aluminium separators, already pressed in the shape of keys were used at the joints.

To form duct, 7 mm Ø H.T. wires were used during casting and these wires were pulled out of the concrete after 24 hours of casting.

2.5 Casting Specimen:

Precasting Operation: After assembling the side-shutterings and the faces of mould, ties were tied at specified intervals. The inner part of the mould was thoroughly oiled and required pre-tied M.S. reinforcement and aluminium separators were placed inside. For duct formation 7 mm Ø oiled H.T. wires were passed through the holes made in faces and separators. The wires were prestressed to some amount of force which would prevent the bending of wires during casting operation and thus leaving a straight duct after it was pulled out.

Nine standard 150 mm cube moulds were oiled inside for taking concrete sample for finding out the compressive strength at the end of 7 days, 28 days and on the day of testing of beam.

Then the materials were weighed for 3 batches of mixing of concrete. For the cement of batch 1 quality, the materials cement - 62 kg, sand - 62 kg, aggregate (12 mm) - 40.2 kg, aggregate (20 mm) - 74.5 kg. were weighed for specimens 1B1S to 7B7S. For specimens 8B7S to 10B7S material weighed were cement - 53 kg, sand-70.5 kg, aggregate (12 mm) - 45.8 kg aggregate (20 mm) - 85 kg.

Pre-casting operation was done on the previous day of the casting of the beam.

Mixing and Placing of Concrete: Concrete was mixed in three batches for each beam. Each batch weighed about 2.5 kN of

concrete. The concrete was mixed in a tilting type revolving drum mixer of capacity 0.15 m^3. Mixing was done for 8 to 10 minutes and then the thoroughly mixed concrete was poured into the moulds and cubes. Careful needle vibration was adopted to remove entrapped air in the concrete.

Duct Formation and Post Casting Operation: After 3 to 4 hours (depending upon the temperature on the day of casting) the grips of 7 mm Ø H.T. wires were removed and wires were rotated about its own axis and then pulled outward and pushed back for several times for about a metre. The pulled portions were oiled on both the ends and again pushed back and forth. This operation was repeated 3 to 4 times in every hour for next four hours. H.T. wires were pulled out on the next day after 24 hours of casting and side shutterings were detached. The cast segments were then covered with wet gunny bags or jute to keep the beam in thoroughly moist atmosphere . Water was sprinkled several times in a day to keep the gunny bags wet all the time.

After 5 days, the segments and aluminium separators were gently removed from the mould base and shifted to nearby place for making the mould ready for casting of the next beam. All beams were cured for 28 days. The cubes were also cured under the same condition as that of beams.

Beam no. 1B1S and 2B1S which were single unit beams were removed from the mould base after 7 days. These beams

were partly prestressed with 5 mm Ø H.T. wires with 15.0 kN jack pressure on each wire. This partly prestressing operation was done before shifting the beam at the end of 7 days.

2.6 <u>Post Tensioning:</u>

60 kN Gifford - Udall hydraulic jack was used for prestressing of wires. For the anchorages of wires, split conical wedges for 5 mm ϕ H.T. wires with cones having tapered hole barrels were used.

First of all,the segments were brought inside the laboratory and arranged on the flat ground in the same sequence as they were cast. The segments were kept 50 to 70 mm away from each other. All the joints were wetted for nearly 1 hour. 5 mm ϕ H.T. wires were inserted into the ducts through all joints. The ends of the wires were cleaned of any rust with emery paper where it was to be anchored. 10 mm thick full size (170 mm x 270 mm) anchor - plates were also placed at the ends of the beam. Anchor cones were gripped on all four wires at one face, keeping nearly 100 mm length of wires protruding out. Thin cement slurry was put on the joints to fill any gap in between joints caused by thickness of aluminium separators placed during casting. Then the segments were joined and properly aligned in a straight line and prestressed from the other face. First of all,one of the top wires was pulled to 15 kN by the jack and

then the other wire, diagonally opposite to it, was pulled. Second wire was pulled to the full prestressing force of 25 kN jack pressure. The jack was moved to the first wire again and it was pulled to full load. Similarly other two wires placed diagonally opposite to each other were also pulled.

After all wires were pulled 5 lines were marked with a pencil near the wedges. The lines were 2 mm c/c and it was done to observe any slip in the wire during test of the beam.

This beam was then placed on the loading frame and white washed. White washing of the beam was done to detect any crack, appearing on the beam, immediately.

2.7 Experimental Set Up:

The plate 2.3 shows the experimental set-up. The two span continuous beam specimens were tested under two point loads, each load on each span at a distance of 1200 mm from mid-support.

The supports of the specimens were so fabricated that it behaved like a hinge support at the centre and roller supports at the ends. Fig. 2.3 and Fig. 2.4 show the details of the hinge and roller supports.

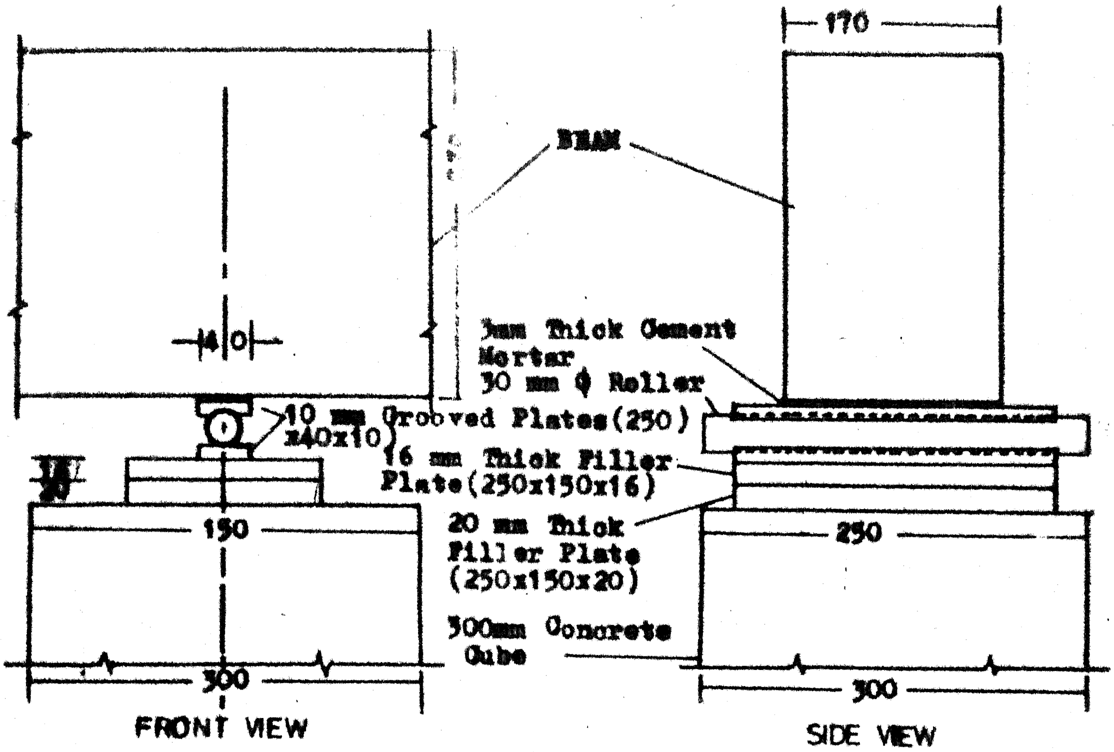


Fig 2.3 HINGE SUPPORT

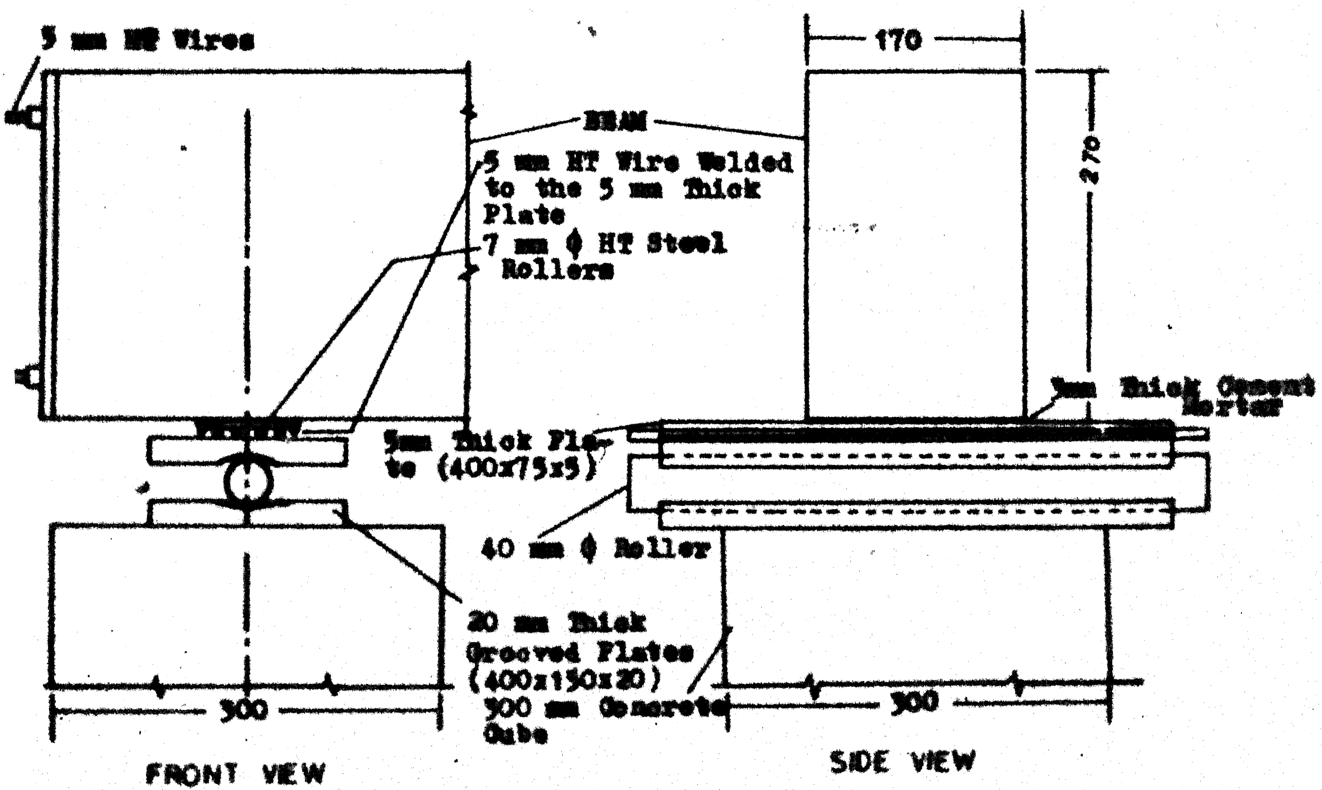


Fig 2.4 ROLLER SUPPORT

The loads on the beam were transmitted by pulsator through 108.8 kN (24000 lb) pulsator jacks, 500 mm apart. These point loads were transferred to two points, 2400 mm apart and equidistant from the centre of the frame or jack-assembly. The transfer of the load points was done by using a 2800 mm long ISMB 250 and steel rollers 50 mm diameter. M.S. plates of 200 mm x 75 mm x 5 mm, 3 numbers of 7 mm ϕ H.T. rollers and 200 mm x 130 mm x 10 mm plates were used one below the other under 40 mm dia roller. 40 mm dia roller was arrested at its position by two angles welded at the base of reaction girder. The plate 200 mm x 130 mm x 10 mm was provided to avoid any concentration of stresses and crushing of concrete, directly under the load. For full seating of plate on the beam 3 mm thick cement mortar was used.

2.8 Testing:

Three sets of beams were cast, viz.,

i) Single unit beams, 2 numbers (1B1S and 2B1S) as shown in Fig. 2.5(a).

ii) 4 numbers of 5 segment beams (3B5S, 4B5S, 5B5S and 6B5S) as shown in Fig. 2.5(b).

iii) 4 numbers of 7 segment beams (7B7S, 8B7S, 9B7S and 10B7S) as shown in Fig. 2.5(c).

One specimen (beam) of each set was tested under static loading to find out the ultimate load capacity of the beam and to compare it with the theoretically computed value.

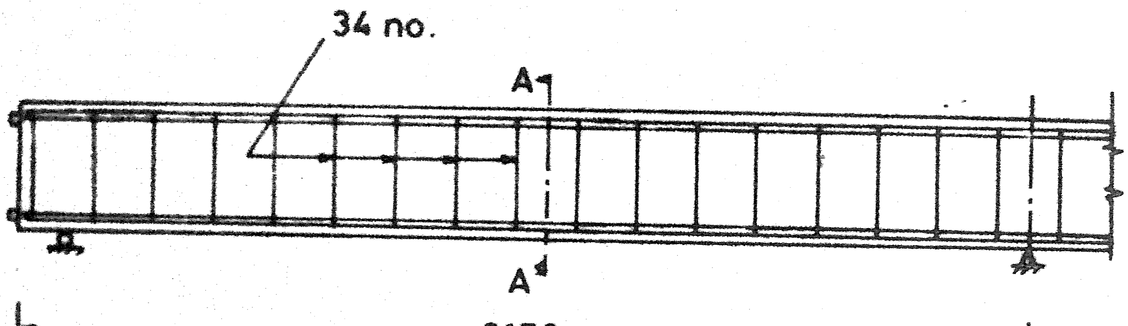


FIG. 2.5 a. Reinforcement details in single unit beam

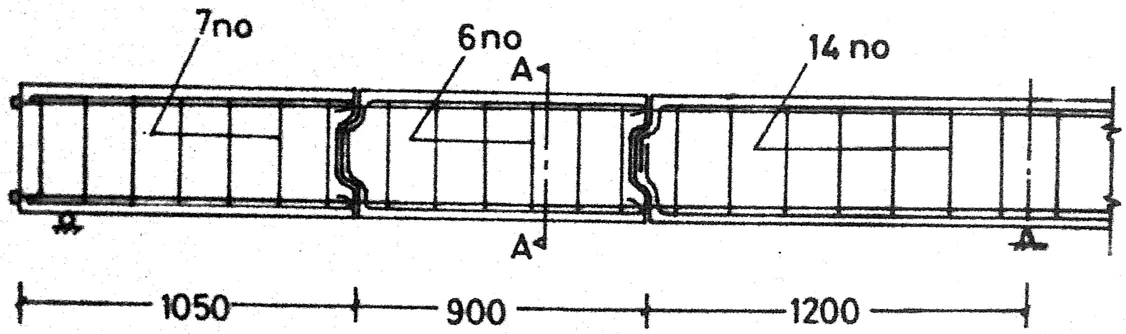


FIG. 2.5 b. Reinforcement details in five segment beam

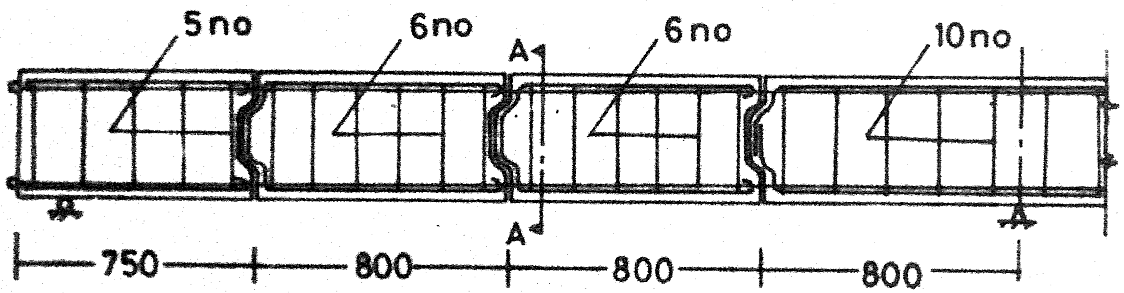


FIG. 2.5c. Reinforcement details in seven segment beam

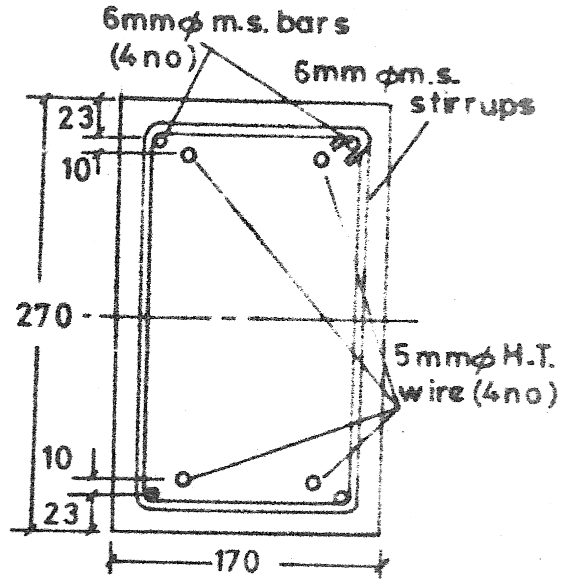


FIG. 2.5d. Section at A-A

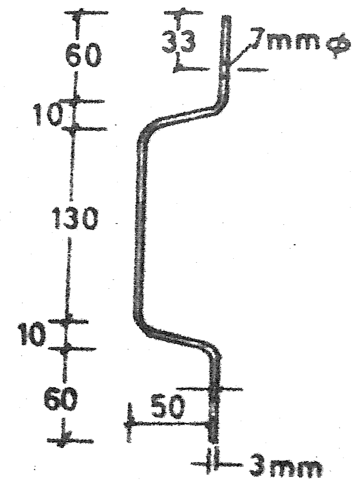


FIG. 2.5e. Aluminium separator

Beam no: 1B1S, 3B5S and 7B7S were tested under static loading condition until it failed. Deflection of the beam was measured at different points with the help of dial-gauges kept under the beam.

All other beams were tested under pulsating loads. At the beginning of experiment with a new beam, it was tested under gradually increasing static load for two cycles; with upper load as peak load and lower load as zero. After each cycle beam was allowed to rest for 5 minutes to recover its strain. Before starting a new cycle,the recovery in deflection and residual deflection were noted. The beam was loaded upto upper load of pulsating loads and set on pulsation.

The lower load or lower bound corresponding to some fraction of ultimate load represents the dead load of the structure. In the present experiment, the lower load was restricted based upon the minimum load which pulsator could go for repeated loading.

The upper load or upper bound corresponding to some fraction of ultimate load represents the sum of the dead load and live load. This was taken as 66 percent, 63 percent and 60 percent of ultimate load in different beams.

The peak load corresponds to occassional load that is likely to come on to the structure and it corresponds to the sum of dead, live and wind or unforeseen load.

The frequency of pulsation was kept as high as possible to 400 cycles/min. Increasing the frequency to about 500 cycles/min. caused the almost resonance condition as the loading frame had the natural frequency of about 500 cycles/min. The beams were put to pulsating loads for about 400 minutes and allowed to relex for about 1040 minutes. The deflections were recorded at no load and at upper load every day, till it failed.

The failure of the beam under pulsating loads due to excessive deflection or crushing of concrete at some part, was automatically recorded by the pulsator and the machine used to get stop. After this, the beam was monotonically loaded until failure under static loading known as 'post-pulsating static test'.

CHAPTER - 3

EXPERIMENTAL RESULTS AND DISCUSSIONS

3.1 Nomenclature

Beams are numbered as nBmS where n represents the serial number of the beam being tested, B indicates beam, m is the number of segments in the beam and S stands for segments. e.g. 6B5S represents the sixth beam with 5 segments.

3.2 Discussion on Controlled Material Testing

It has been already discussed in Chapter 2 that nine cubes were cast along with each beam to know the compressive strength of the cube at the end of 7 days, 28 days and on the day of testing of the beam.

The compressive strength of cube on the day of testing of beam was found out to know the property of the beam section, viz., under-reinforced or over-reinforced. On the basis of this, the theoretical strength of the beam was calculated.

The 7-day cube strength was found out to predict the strength of the cubes at the end of 28 days. Beams 1B1S and 2B1S, which were single unit beams, were prestressed partly at the end of 7 days of casting and they were removed

from the mould base. Before prestressing the beams, the required cube strength of the concrete was checked with the 7-day strength of cubes.

Beams were brought to the laboratory after 28 days of casting and they were prestressed. Before prestressing fully, it was essential to know the cube strength of concrete.

Three specimen of prestressing wire were tested under gradually increasing static load to know the failure load. Stress-strain curves were plotted to know the proof stress, modulus of elasticity, etc.

Three specimen of prestressing wire were also tested under repeated loading between 50 percent and 70 percent of the ultimate load of the wire. It was found that specimen failed at about 372,000 cycles.

Three specimen of 6 mm ϕ M.S. bars were tested under gradually increasing static load to find out the yield strength and the ultimate strength of the bar. Stress-strain curves were plotted.

Tests on controlled cubes, controlled wires and M.S. rods were conducted to study the behaviour of the materials and the beam-specimen separately. If beam had withstood more than 400,000 cycles of pulsating loads then, one could expect the fatigue failure of wires one after one. Then, immediate strength of the beam after failure of one or more

wires before the final collapse of the beam could have been studied. If the beam fails before 400,000 cycles, then the mode of failure, amount of moment redistribution, and crack propagation during pulsating loads etc. could be studied.

The deflection of the beam was measured at three points on each span with the help of dial gauges. The dial gauge positions are shown on load-deflection curves (Fig. 3.1 to Fig. 3.11). The approximate maximum deflection of the beam was measured just before the collapse with the scale.

3.3 Static Load Test:

Three beams, namely 1B1S, 3B5S and 7B7S, had been subjected to gradually increasing static load. The idea of conducting static load test of a specimen from each set was to see the variation of ultimate capacities of the specimen with regard to the number of joints in the specimen, amount of moment taking place and to estimate upper and lower load for testing other identical specimen under pulsating loads.

In Table 3.1 theoretical collapse load of specimen, its observed collapse load, observed cracking load and maximum deflection observed at 80 percent of observed collapse load are tabulated.

Beam 1B1S:

(i) The ultimate load observed was 96.35 kN and the theoretical ultimate load for bonded tendons is 84.56 kN. The beam was over-reinforced theoretically. Since the beam was unbonded, so it should give a smaller collapse load by 10 to 20 percent but it gave a higher value by about 14 percent.

(ii) Linear load-deflection pattern was observed till 45 percent of the ultimate load.

(iii) There was a return of dial-gauge readings in DL-1, DL-2 and DL-3 between 56.5 percent and 63.5 percent of collapse load.

(iv) The tension crack at the mid-support was located about 120 mm away from the centre and other crack was about 100 mm away from the nearest load point.

(v) Inward translation of one of the roller supports was 20 mm horizontally.

(vi) One prestressing wire at the bottom broke at the failure of the beam.

(vii) The cracks appeared on the full depth of the beam and it was difficult to say anything about the actual depth of the tension crack. Evidently, this had definitely reached compression zone of the specimen. Crack width was approximately 30 mm under one of the load points. At the central support, a concrete wedge was formed due to two cracks

10 and 13 mm wide and 160 mm apart at top which met at a point and some depth.

(vii) Load deflection pattern showed strain hardening of the material of the beam after 64.6 percent of collapse load.

Discussion:

(a) Because of large deformation due to formation of mechanism and smaller duct diameter, there may be friction which predominates between wire and duct. This frictional force gives rise to the development of local stress between the two links, which increase the value of collapse load.

(b) As such there may be coefficient of variation of 5 percent in the strength of steel and a variation of 15 percent in concrete. There may be variation in the dimension of section and the span length of the beam since supports were displaced after every testing. The variation in span length causes variation of the load positions and also they may not be placed symmetrically between the joints. So, observed collapse load may be higher than theoretical collapse load because of inherent error caused due to statistical variations.

(c) Because of the formation of the first plastic hinge a little away from the centre, there may be unsymmetry in the beam with respect to load positions. This may cause increase in deflection on one side much more than that on the other side. This particular phenomenon causes decrease

in flexural stiffness on large deflection side and so the crack widens. At this stage, redistribution of moments also takes place and weaker section gets more moment causing more stress in wire. The stressed wires pull the support towards itself at the section leading to closing of crack on the other side. Hence, the section on smaller deflection side stiffens.

The above discussion can be justified by the load-deflection plot which shows a very little net increase in deflection in one span between 56.5 percent and 63.6 percent of collapse load. But the actual observation states that there was some increase even after 56.5 percent of ultimate load which reduced because of gradual stiffening. This particular phenomenon can be said as readjustment also.

Beam 3B5S:

(i) The observed collapse load of the beam was 76.20 kN and theoretical collapse load of the beam for bonded construction is 73.20 kN.

(ii) The cracking load was observed to be 42 percent of the collapse load.

(iii) Load-deflection plot was linear till 45 percent of the collapse load.

(iv) Load points were the critical points as they were placed directly over the joints. Flexural failure took place at the joint, with joint opening of the order of 40 mm and

crack width of about 12 mm,at the centre at mid support. The widths of crack and opening were noticed after the beam was unloaded.

(v) Load-deflection curve showed strain-hardening in the beam between 71.4 percent and 85.7 percent of its ultimate load which is not realistic in most of the structures.

Discussion:

(a) The reason for the value of observed collapse load to be about 4 percent higher than the theoretical value can be explained in the same manner as for beam 1B1S.

As such the theoretically estimated collapse load was also not exact,for the losses due to the number of joints, particularly cement-slurry joint,was not taken into account, which would have decreased the value of theoretical collapse load.

(b) The observed strength of unbonded beam being more than the theoretical strength of bonded beam may be explained in some other way also. The assumption that there developed average stress in the unbonded tendons may be partly correct as the wire may get arrested between the kinks both at joints and cracks by friction which predominates. So, the frictional force increasesthe value of observed load. This frictional force ceases after certain load and beam behaves inelastically.

(c) In this case also, it was found that after certain load level, these was sharp increase in deflection on one side compared to the other side. This may also be explained in the same way as has been discussed for beam no: 1B1S.

(d) Full moment redistribution had taken place as observed collapse load is more than theoretical load.

Beam 7B7S:

(i) The observed collapse load of the beam was 72.28 kN as compared to 85.24 kN, theoretically computed value for bonded construction. If it is assumed that unbonded beam has the strength 10 to 20 percent less than its corresponding bonded beam, then its collapse load would lie between 68.2 kN and 76.72 kN.

(ii) 12 mm wide crack opening at the top at central support of the beam and 14 mm joint opening at the bottom of the joint number 2 (joints were numbered 1, 2, ..., 6 from left to right) were observed.

(iii) Beam failed under secondary crushing of concrete at the joint.

(iv) Linear load-deflection curve till 54.5 percent of collapse load was observed.

(vi) Inelastic behaviour of the beam after 60 percent of collapse load and no strain hardening were noticed.

(vii) Maximum deflection at collapse load was observed to be approximately 109.0 mm.

Discussion:

(a) Beam showed better characteristic than beam no:1B1S and 3B5S, as there was no strain hardening and longer range of linearity in load-deflection curve. This might be because of richer mix of the concrete which was not obtained in the case of beams 1B1S and 3B3S. Concrete with richer mix shows a greater elastic range than an inferior mix. Though the beams were under-reinforced, quality of concrete might have some effect.

(b) There is decrease in collapse load as compared to beams with 5 segments or monolithic beam. As segment size goes on decreasing with the increase in number of segments there occurs more readjustment of segments and joints while loading, causing loss of prestress force, hence there is decrease in the total load carried by them.

(c) There was no M.S. reinforcement continuing through the joints and also segments were held together by dry-joints, i.e., no bonding between the segments. So, there is discontinuity in the tensile stresses at different lengths at the bottom in the beam. This also causes decrease in strength.

(d) The beam might have collapsed before full moment redistribution of moments because of large joint opening due to average strain of the unbonded wire and limiting strain in concrete.

(e) From the crack pattern it was obvious that it was a combined flexure and shear failure, so stirrups should be placed close enough near the joints. If possible, M.S.bars may be continued through the joints by welding i.e.,cast in place joint is advised.

3.4 Pulsating Load Test:

Seven beams were tested under pulsating load. Table 3.2 gives a brief picture of the test results.

Based on the test results obtained from static test, the beams are assumed to have lower collapse load when set on pulsating. The following expression was assumed.

$$P_{tu} = k\, P_{to}$$

where P_{tu} = Theoretical collapse load of the beam for unbonded section with due moment redistribution factor.

k = Moment redistribution factor
= 1.0 for single unit beam
= 0.95 for 5-segment beam
= 0.85 for 7-segment beam

P_{to} = Theoretical collapse load of beam for bonded tendons and full moment redistribution.

All-beams were tested under gradually increasing static load before the beam was put on pulsating loads. Two cycles of static loading were done with upper load normally upto 45 to 60 percent of P_{tu} and lower load to zero. The main reason of

testing the beam under static load for two cycles was to see whether any slip at the anchorage took place. There might have been readjustment of segments over the joints, which may cause heavy loss in prestressing force. The beam could be restressed if it was found that there was loss in prestress due to readjustment of segments.

Beam 2B1S:

The beam had cracked at two sections as shown in the Fig. 3.2 while handling,before it was placed on the loading frame. Instead of abandoning the beam, it was restressed and placed on the loading frame.

(i) The load-deflection curve was linear upto 58 percent of the theoretical ultimate load, P_{tu}.

(ii) The load-deflection curve seemed to be very peculiar in the first cycle of loading.

(iii) Upper and lower loads for repeated loading were 61.0 percent and 36.6 percent of P_{tu}.

(iv) Larger deflection was noticed at one load point and recovery of deflection on the other was observed under pulsating loads.

(v) M.S. rods on the tension side of the section broke under pulsating loads at the mid support and at one loading point. After failure of every M.S bar there was gradual increase in deflection of the beam.

(vi) The beam did not fail at fractured locations and fractures did not open even.

Discussion:

(a) The load deflection curve in the first cycle is very much peculiar because of readjustment of the broken parts of the beam. Since the same nature of curve did not repeat again in the second cycle so above conclusion may be right.

(b) M.S. reinforcement failed due to fatigue as it was subjected to large stress-range. So, non-stressed high strength steel should be provided in place of M.S. steel.

(c) Beams failed at critical points only, so there was no danger because of the mistake in handling.

Beam 4B5S:

(i) The beam was loaded upto 90.1 percent of P_{tu} in the first cycle. This load was said as peak load or occasional load. The crack width was of the order of 1 mm to 2 mm at the top at mid-support.

(ii) The beam was restressed and set on for two more cycles of static loading upto 77.7 percent of P_{tu}. Then it was set on pulsating loads between 62.1 percent and 34.2 percent of P_{tu}. The beam withstood 1540 cycles.

(iii) Breaking of M.S. rods at the top of central support was noticed.

Discussion:

The aim of loading the beam upto 90.1 percent of P_{tu} as occassional load was to study the behaviour at this load and how the property of the beam could be improved if it gives large residual deformations, whether restressing is useful to a great extent or some other methods also should be adopted along with restressing, etc.

Beam 5B5S:

(i) Deflection is linear upto 55.4 percent of P_{tu} in pre-pulsating static test. The maximum load subjected on the beam was 72.6 percent of P_{tu} in the pre-pulsating test.

(ii) This beam was set on pulsating loads between 0.726 P_{tu} and 0.363 P_{tu}.

Discussion:

(a) The main reason of early (premature) failure of the beam may be due to larger deflection which was already shown under two cycles of pre-pulsating static loads. The value of upper load under pulsation was also large.

(b) The strength of concrete was weaker in case of 5B5S. It was only 28.8 N/mm^2, which has lower tensile strength and smaller range of elasticity and linearity. In every loading

concrete reaches its yield point at extreme fibre and it results in early failure of beam. The AASHTO (17) specification states that minimum cube strength shall be not less than 35 N/mm^2 for post tensioned segmental construction.

Beam 6B5S:

(i) Two cycles of pre-pulsating static loads were applied. Maximum load level was 0.684 P_{tu}.

(ii) Upper load and lower load for pulsation were 0.684 P_{tu} and 0.342 P_{tu} respectively. The beam withstood more than $2x10^5$ cycles. It showed a better performance than other beams.

Beams 8B7S, 9B7S and 10B7S:

In these beams the upper loads were 0.60 P_{tu}, 0.571 P_{tu} and 0.541 P_{tu} respectively. Lower loads remained same at 0.36 P_{tu} for all three beams.

(i) Number of cycles the beams survived for were 69,070; 277,280 and 297,490 cycles respectively.

(ii) The load-deflection curve of 9B7S was seen to be linear upto 46.5 percent of P_{tu} and in other two beams linearity were not observed at any stage.

(iii) All beams failed at joints and mid-support. Under pulsating loads, the diagonal crack near the joint went on propagating. This propagation of crack caused spalling of part of beam at joint.

(iv) Propagation of tension crack was observed at the mid-support under pulsating loads.

Discussion :

(a) The beams withstood large number of cycles when upper load was not more than the cracking load.

(b) With increase in number of joints there increases the chances of failure by stress-concentration.

(c) Linearity was obtained in the load-deflection curve only for small range of loads; so with the increase in number of segments, the range will still go on decreasing and hence large factor of safety has to be introduced.

(d) Closer stirrups should be provided near the joints to avoid the failure by shear.

General Discussion:

(i) It was observed that when the upper load was kept higher than the cracking load, the number of cycles under pulsating loads were small. If the upper load was in the range of cracking load, the number of cycles a beam survived is reasonable.

(ii) In all cases M.S. reinforcement on tension side failed in fatigue resulting initially sudden and then gradual increase in deflection.

(iii) Continuous crack propagation from top to the bottom at the centre was observed.

(iv) The final maximum deflection was observed to be between 17 and 29 mm (span/defl. = 176 to 105 approximate), which was more than the limiting deflection. So beams should be designed to restrict the deflection.

Post-Pulsating Static Load Test:

The load causing large deflection in the beam was assumed to be the collapse load. When the reaction girder placed on the beam touched the beam at some point, the beam was said to have undergone large deflection.

It was observed that the absolute value of load carried by each beam was approximately same whether it was single unit beam or beams with 5 or 7 segments. However, the collapse loads were observed to be different in static test.

Discussion:

It has already been discussed that M.S. reinforcement on tension side broke under the pulsating loads at critical locations. Formation of crack and breaking of M.S. reinforcement under pulsating loads render all beams identical with discontinuity of tensile stress at the openings. So the load carried by them is the same.

TABLE 3.2 : PULSATING LOAD TEST

Beam No.	Cube strength N/mm^2	Cracking load kN	Cracking load $\frac{P_{cr}}{P_{tu}}x100$	Lower Load $=\frac{P_1}{P_{tu}}x100$	Upper Load $=\frac{P_2}{P_{tu}}x100$	Ratio $=\frac{P_1}{P_2}$	No.of Cycles	Residual Defl.	$\frac{\text{Span}}{\text{Defl.}}$	Post-Pul. Col. Load kN	Approx. Maximum Defl.at Col. mm	$\frac{\text{Span}}{\text{Defl.}}$
1	2	3	4	5	6	7	8	9	10	11	12	13
2B1S	49.7	60.3	68.3	36.6	61.0	60.0	140,580	19.42	153.7	63.1	62.5	48.0
4B5S	33.2	-	-	34.2	62.1	55.1	1,540*	-	-	-	-	-
5B5S	28.8	43.55	62.1	36.3	72.6	50.0	45,420	21.99	136.4	63.1	109.0	27.0
6B5S	34.9	47.87	63.4	34.2	68.4	50.0	206,650	21.19	141.6	65.3	116.1	26.0
8B7S	39.4	42.43	58.6	36.0	60.0	60.0	69,070	28.50	105.3	64.2	101.5	30.0
9B7S	36.6	43.55	60.1	36.0	57.1	63.2	272,830	16.71	171.5	60.9	187.7	16.0
10B7S	36.8	39.17	54.1	36.0	54.1	66.6	297,490	21.28	141.0	69.6	114.5	26.3

* The beam 4B5S was subjected to a peak load of 90.1 percent of theoretical collapse load.

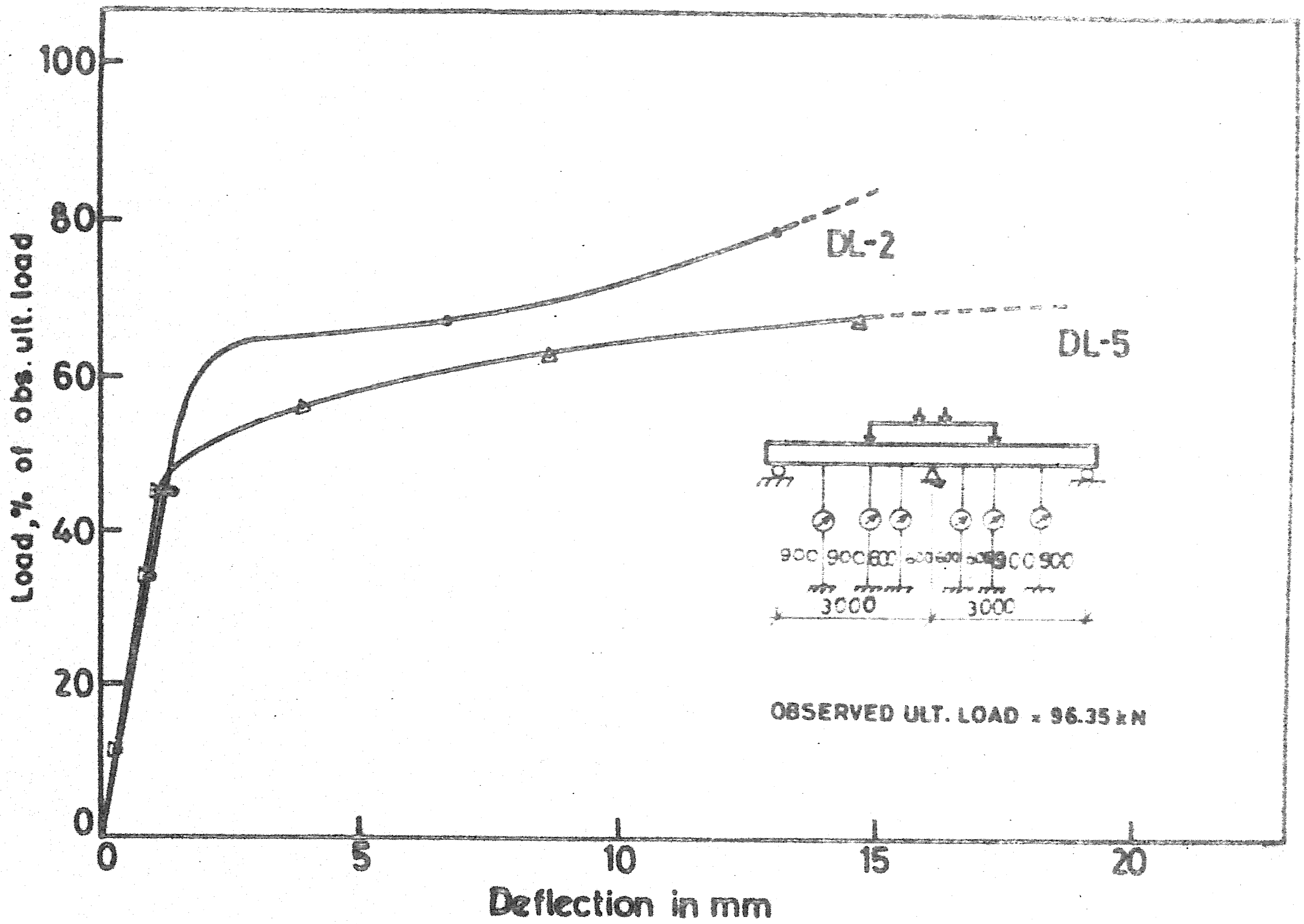


Fig 3.1 LOAD-DEFLECTION CURVE OF BEAM 1B1S UNDER STATIC LOAD

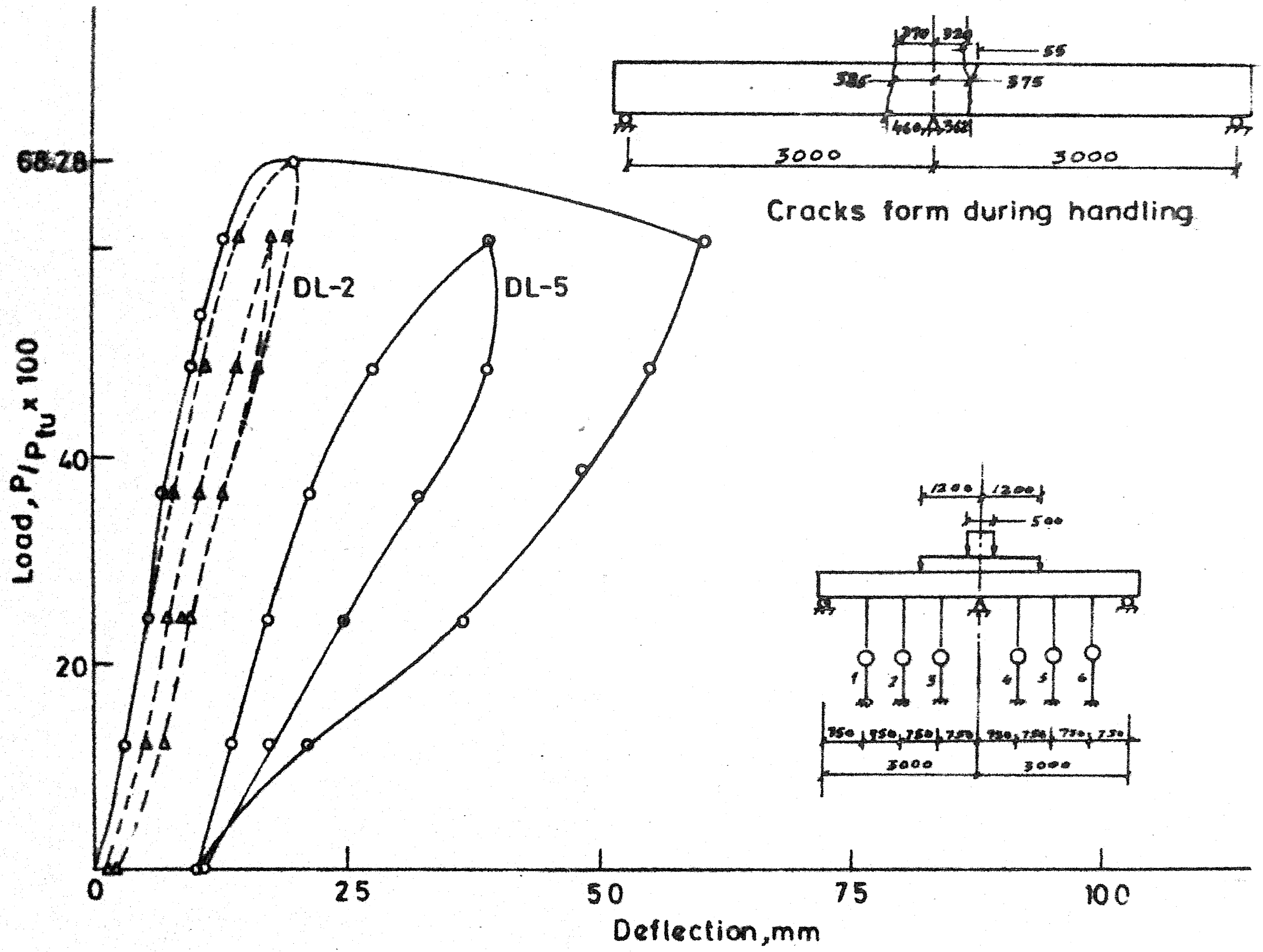


FIG. 3.2 LOAD-DEFLECTION CURVE OF BEAM ZB1S IN PRE-PULSATING STATIC TEST

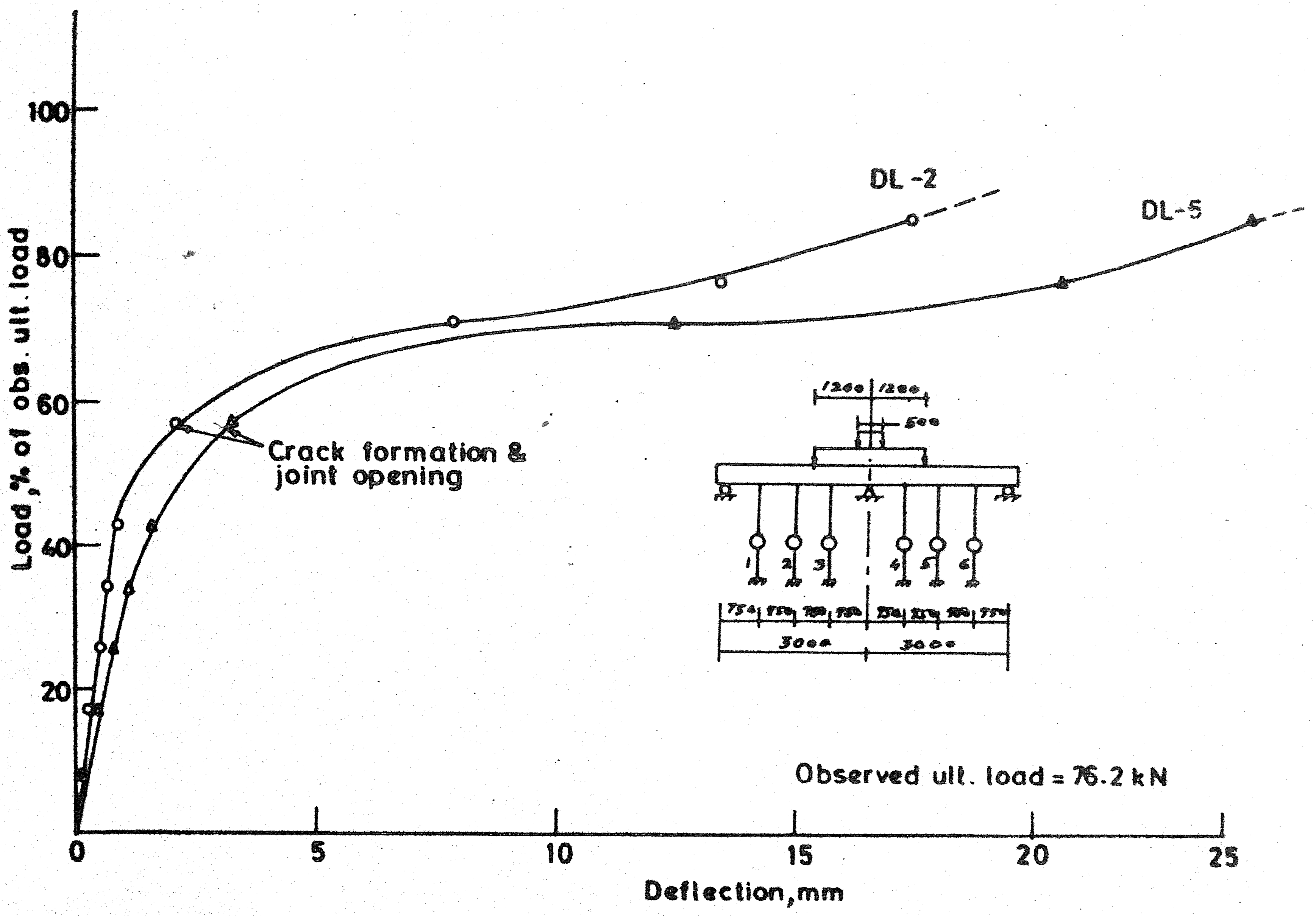


FIG. 3.3 LOAD-DEFLECTION CURVE OF BEAM 3B5S IN STATIC TEST

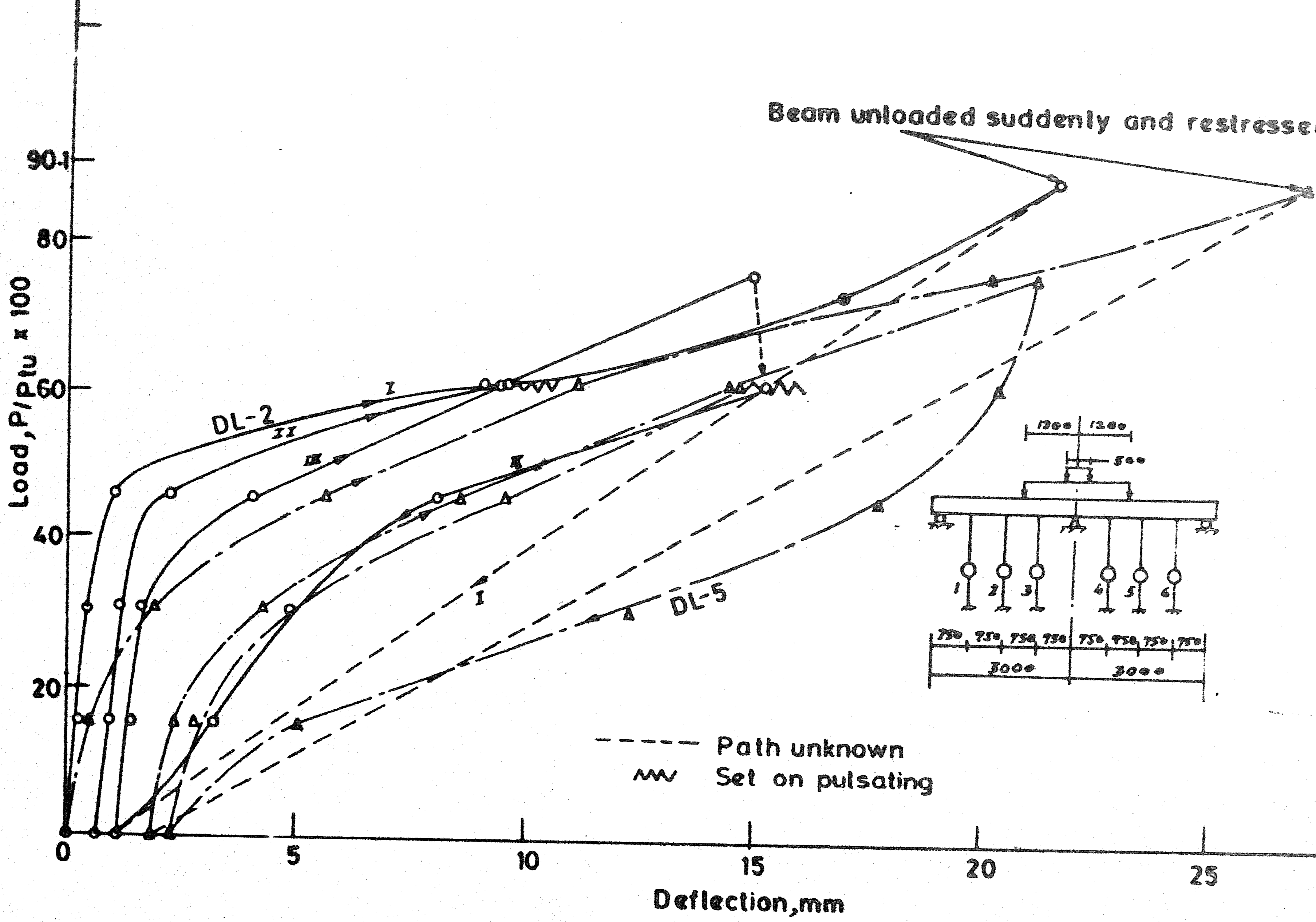


FIG. 3.4 LOAD-DEFLECTION CURVE OF BEAM 4B5S IN PRE-PULSATING STATIC TEST

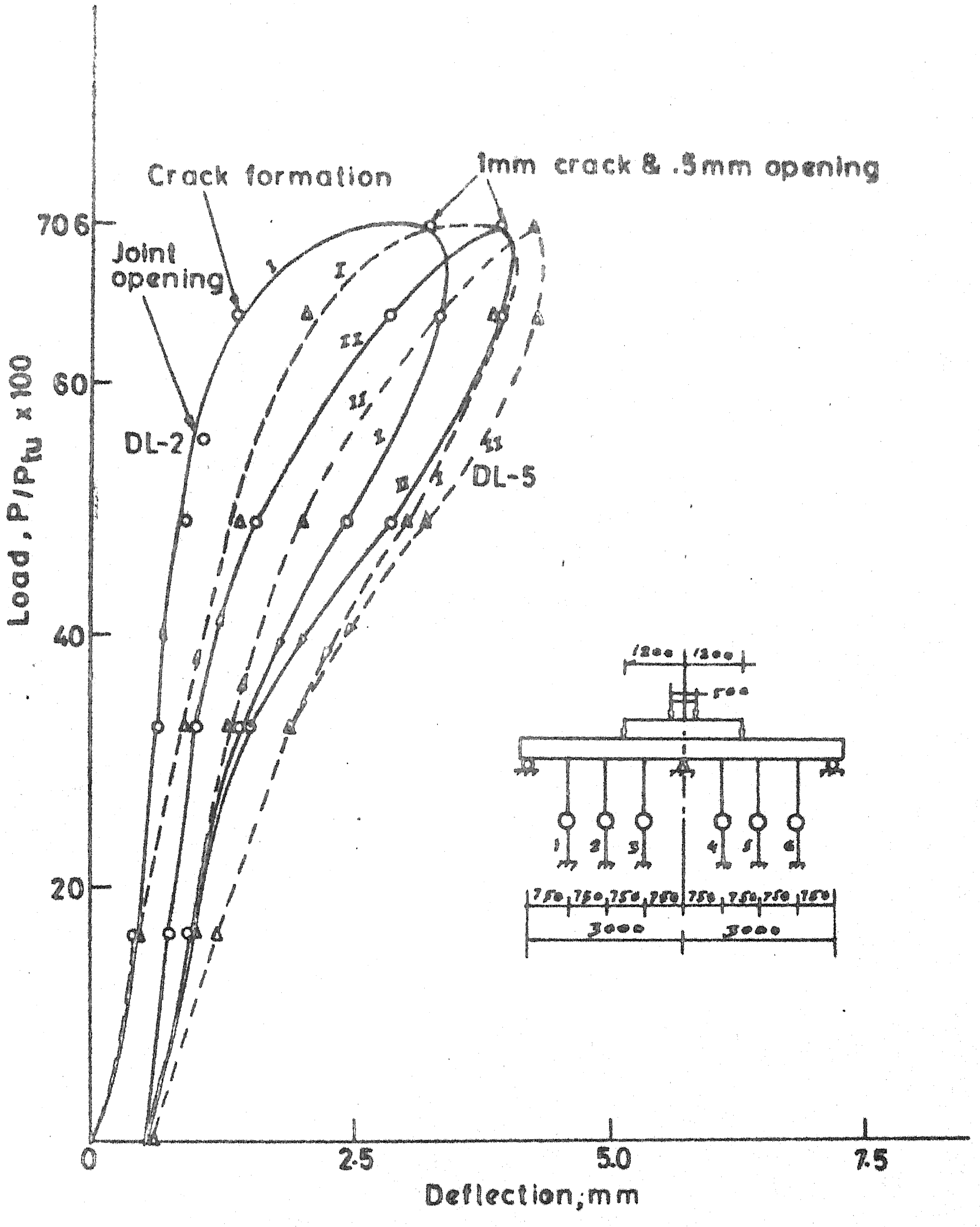


FIG 3.5 LOAD-DEFLECTION CURVE OF BEAM 5B5S IN PRE-PULSATING STATIC TEST

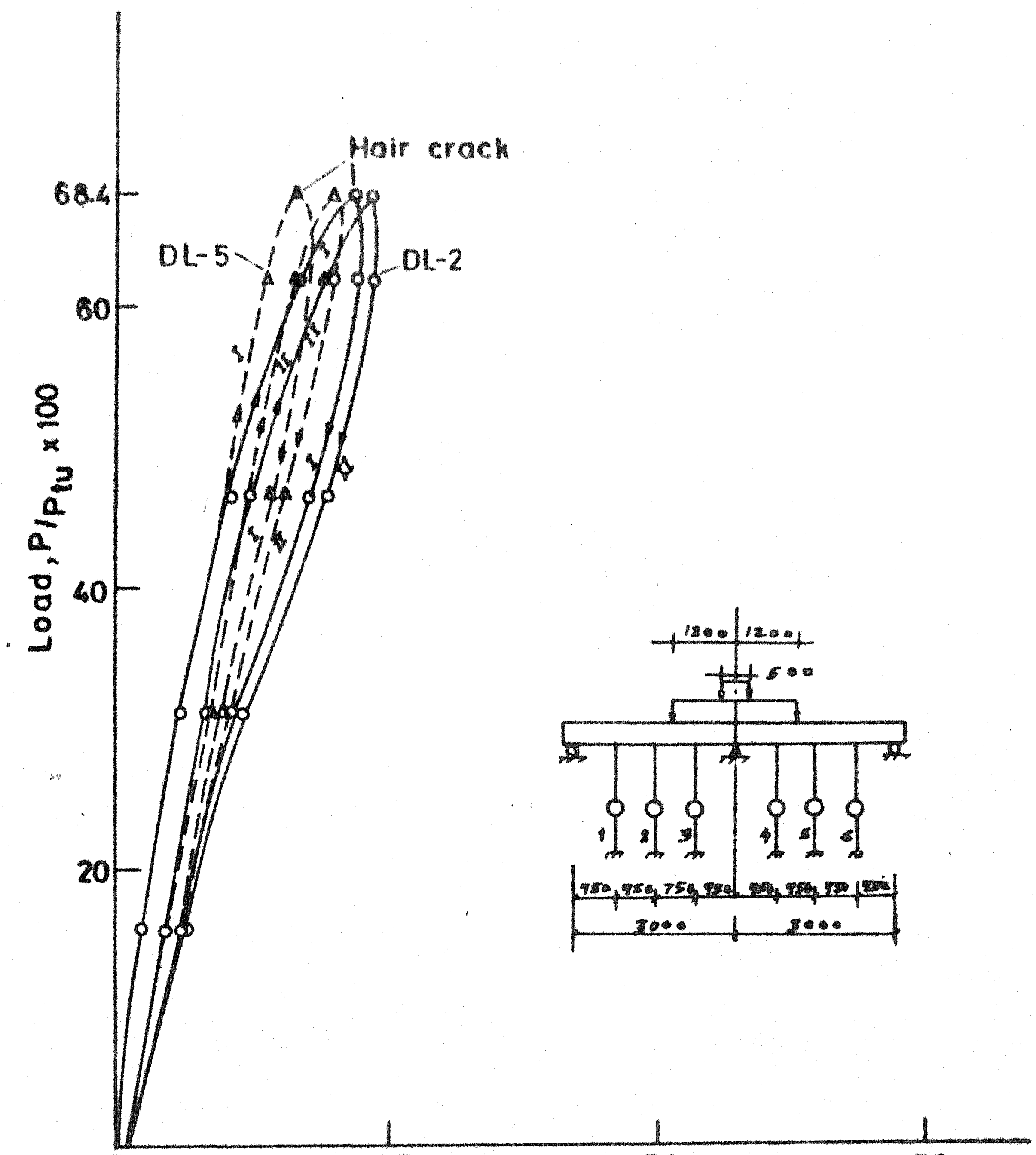


FIG. 3.6 LOAD-DEFLECTION CURVE OF BEAM 6B5S IN PRE-PULSATING STATIC TEST

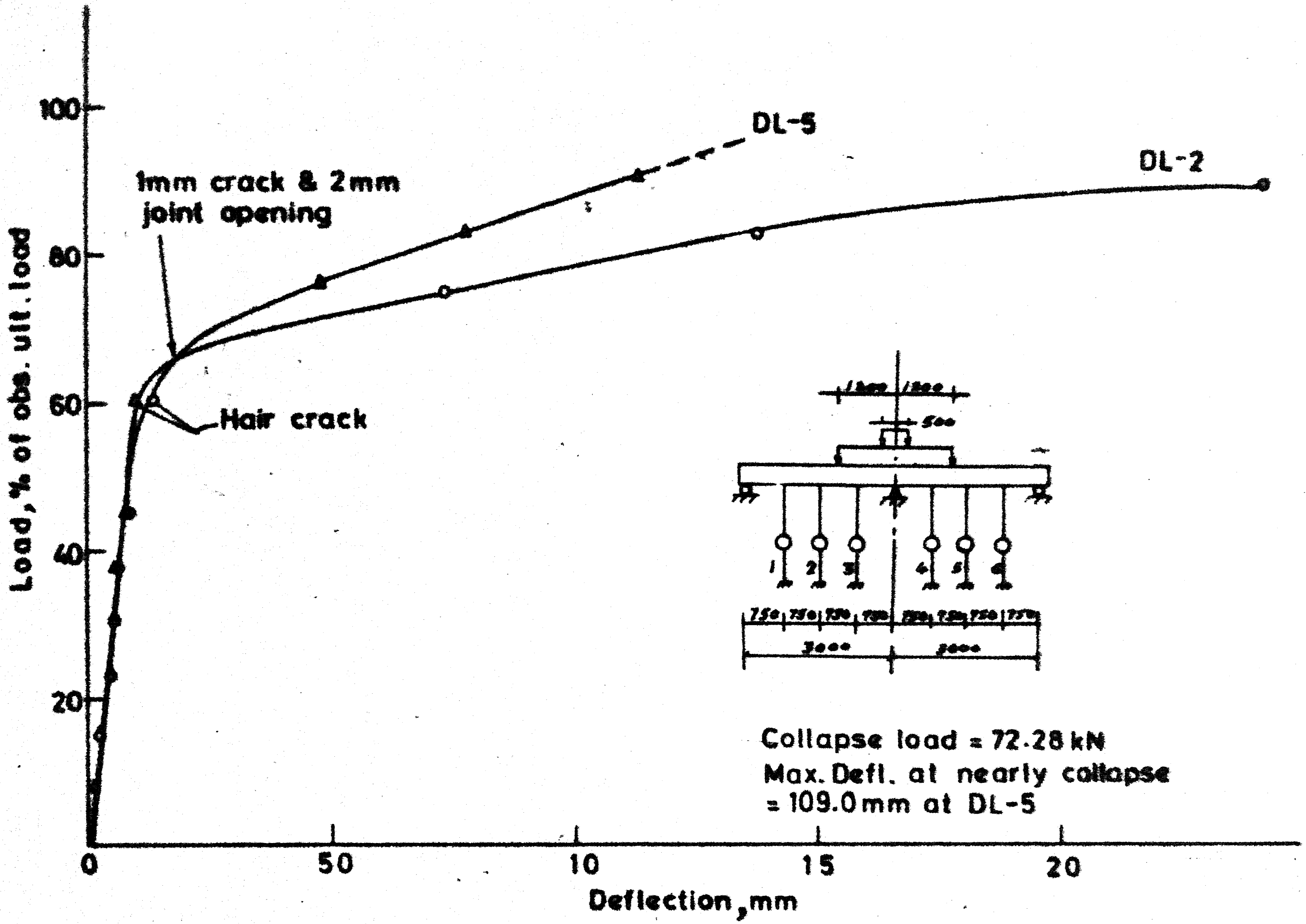


FIG. 3.7 LOAD-DEFLECTION CURVE OF BEAM 787S IN STATIC TEST

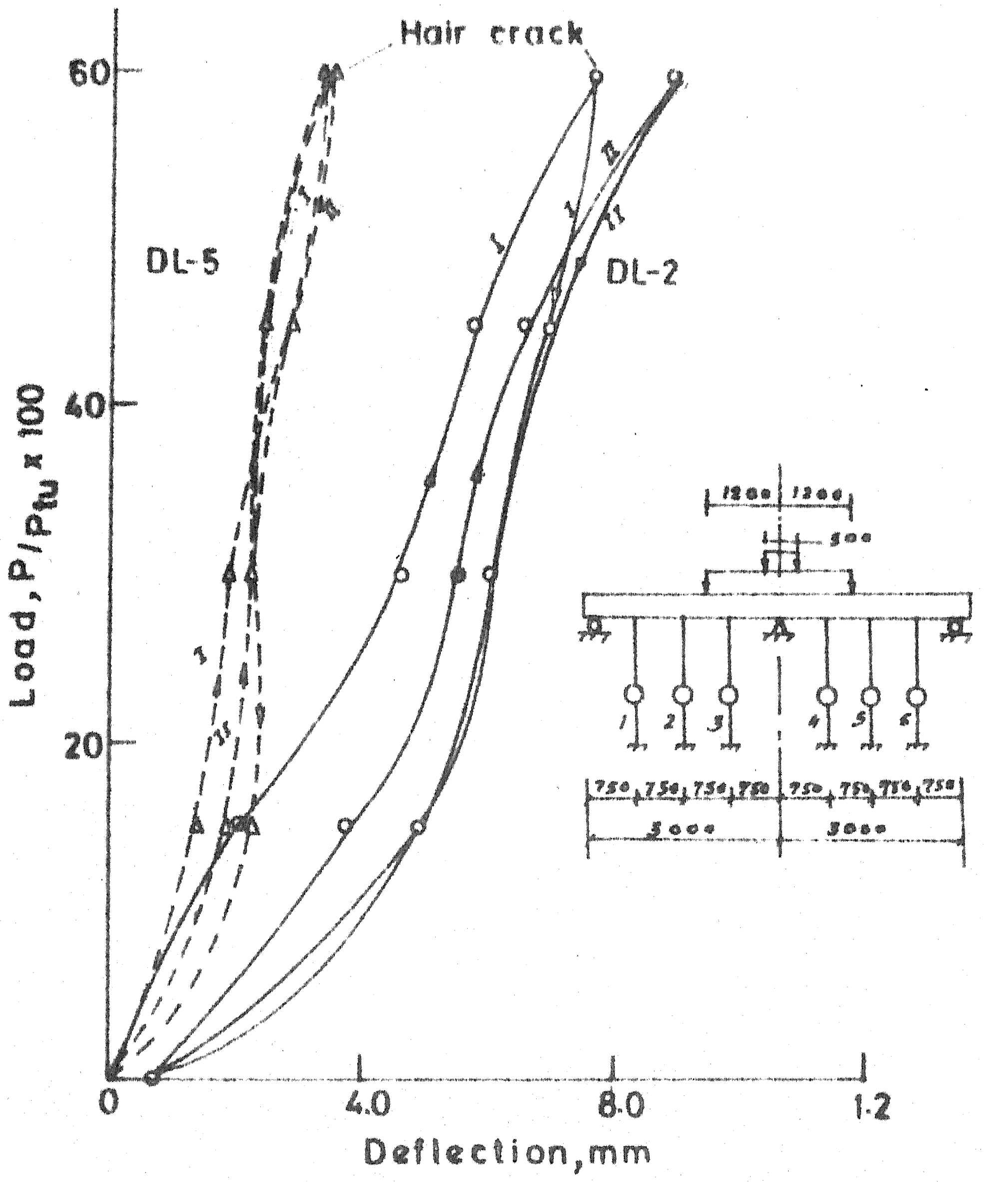


FIG. 3.8 LOAD-DEFLECTION CURVE OF BEAM 8B7S UNDER PRE-PULSATING STATIC TEST

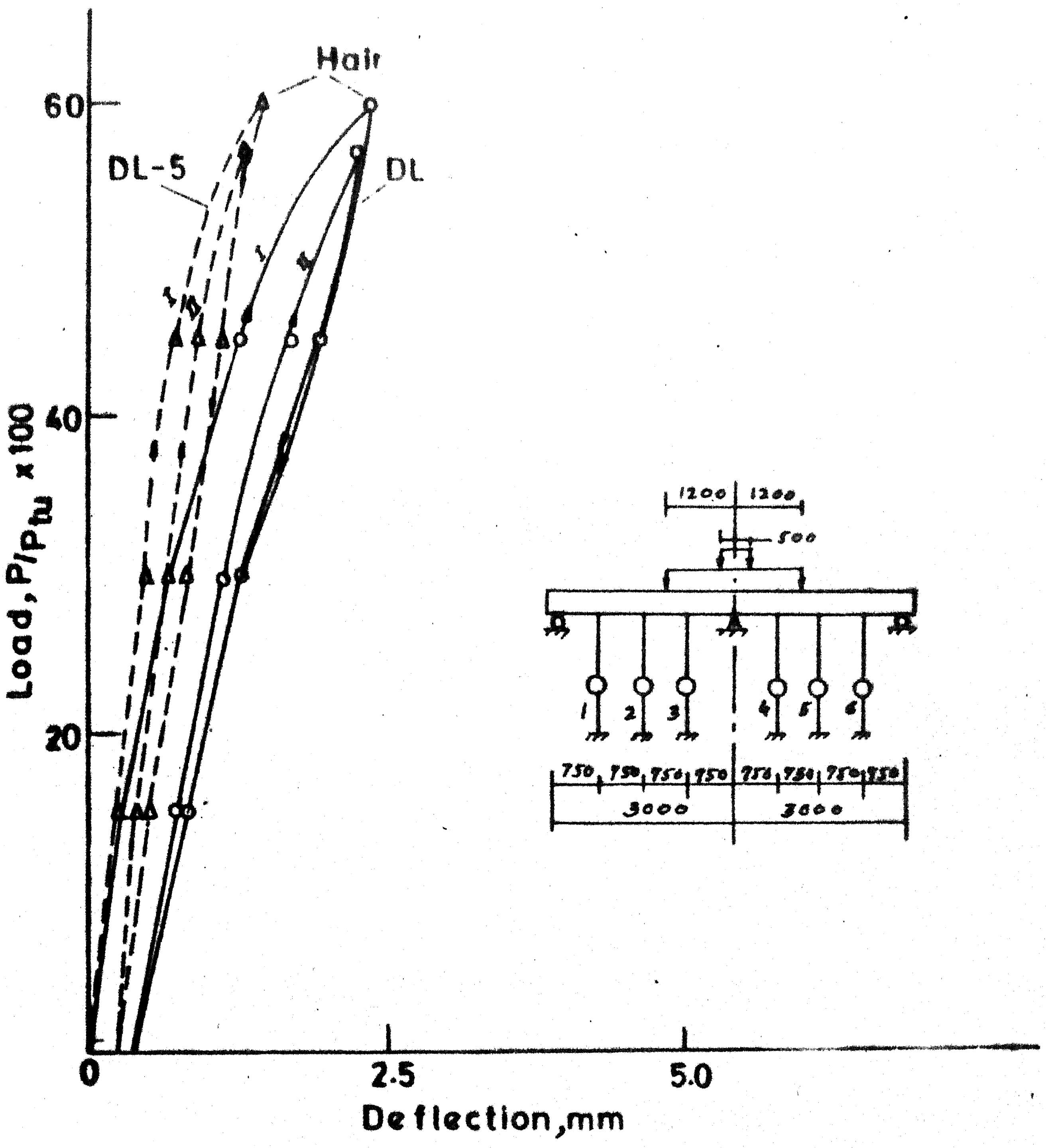


FIG. 3.9 LOAD-DEFLECTION CURVE OF BEAM 9B7S IN PRE-PULSATING STATIC TEST

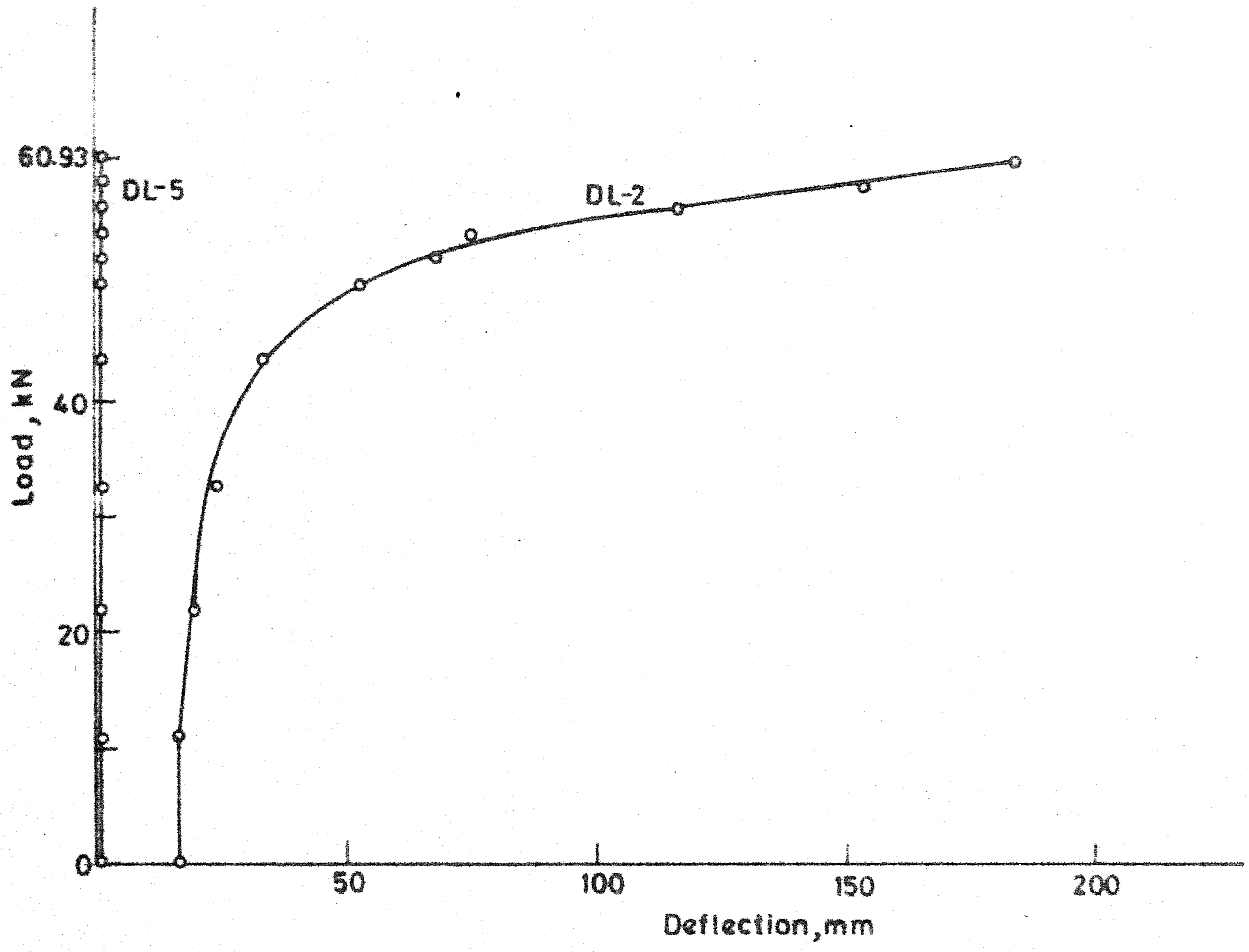


FIG. 3.10 TYPICAL LOAD-DEFLECTION ON CURVE OF BEAM 9B7S POST PULSATING STATIC TEST

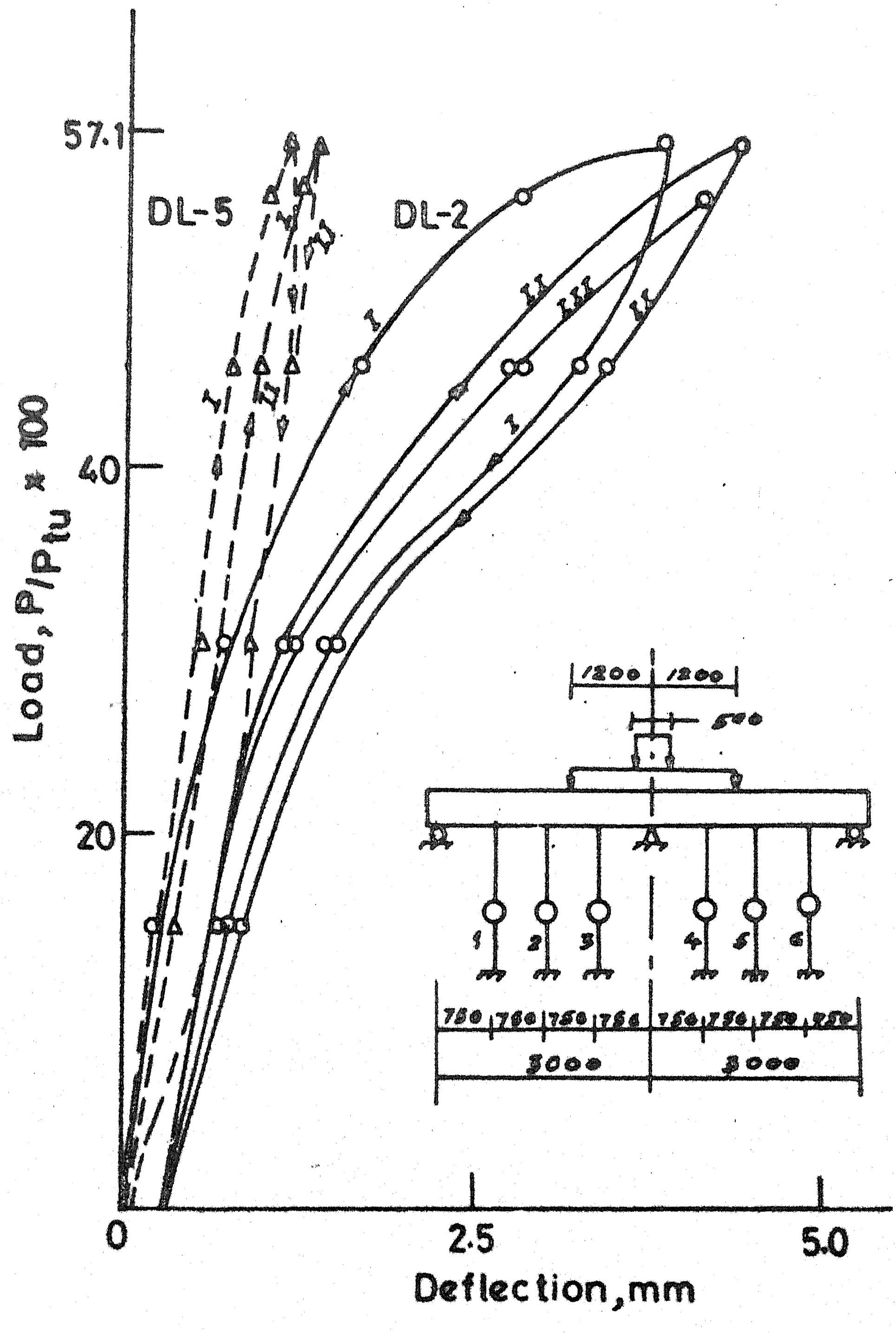


FIG.3.11 LOAD-DEFLECTION CURVE OF BEAM 10B7S IN PRE-PULSATING STATIC TEST

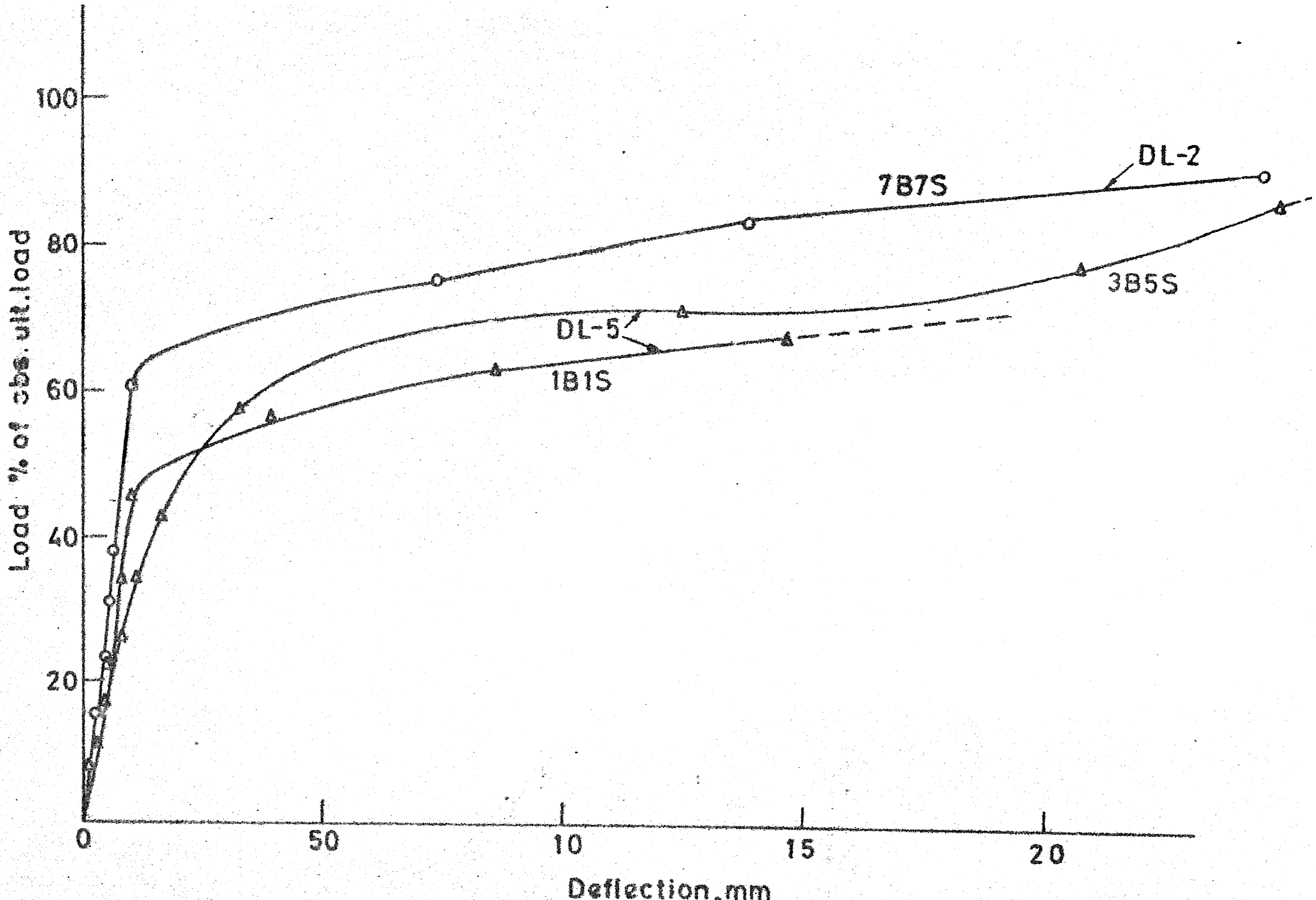


FIG. 3.12 VARIATION IN LOAD-DEFLECTION CURVES OF BEAMS IN STATIC TEST

CHAPTER- 4

CONCLUSION

4.1 Static Load Test:

1. The cracking loads of the segmental as well as the monolithic beams remained between 54.1 percent and 68.3 percent of the theoretical ultimate loads. They are practically the same in magnitudes in the case of all segmental beams and it can also be said that the values will remain the same for monolithic beams also if the M.S. reinforcement is discontinued at the critical locations. So, the nature of cracking load is not much affected by the segmental construction.

2. At the cracking load, the unstable cracks are developed at the critical locations. This was noticed by the jerk or decrease in the load shown by the pulsator. Increase in the deflection under constant load was shown by the dial gauge also. The cracks terminate after some times if the loads are kept constant. Cracks keep propagating and reach the compression zone under pulsating loads.

3. The measured collapse load of the beams decreases with increase in member of segments even though the theoretical values are supposed to be same. This may be due to the fact that full moment redistribution does not take place in beams which is due to averaging of stress in

the tensioned wires, larger joint openings (larger deflection) and limiting strain in concrete.

4. With the increase in number of segments in the beams, the deflection of failure increases themendously. So, adequate design provision should be made.

4.2 Pulsating Loads Test:

5. Sinha (10) tested simply supported beams under pulsating loads. He kept the upper load at 66 percent of the ultimate load for bonded construction. The beams were subjected to peak loads of 75 percent of ultimate load for several times. All the beams survived for more than one million cycles and all cracks terminated at certain point after few hundred thousand cycles. He observed the maximum cumulative deflection of the order of $\frac{\text{span}}{320}$ under repeated loads. This ratio is not very far off from the allowable value of $\frac{\text{span}}{350}$. He, therefore, predicted that there is no danger to the structures under repeated loads.

In case of continuous beams. the maximum cumulative deflection observed was 28.5 mm which is approximately $\frac{\text{span}}{105}$ and it is much greater than $\frac{\text{span}}{350}$.

The main difference between the simply supported beams and that of continuous beams can be explained. There is only one plastic hinge formed in a simple beam for collapse. In

case of continuous beams atleast two plastic hinges have to be formed depending upon the degree of indeterminacy and collapse mechanism. Moment redistribution takes place between the section where plastic hinge has already formed and other sections. Because of the unbondedness of the prestressed wires and of limited number of large cracks, the total strain in the wires becomes large consequently the moment capacity decreases.

6. If the upper load, in the case of pulsating loads, is very close to the cracking load of the beam then the beam withstands approximately 0.2 to 0.3 million cycles. In case of beams which were subjected to upper loads higher than the cracking loads, the beams could withstand only two to seventy thousand cycles of loads.

7. As the number of segments increases in the construction of beam, there is a free rotation of joints about its edges. This results in local stress concentration at the joints. So following methods can be adopted to prevent the above said failure.

7a. Stirrups should be well designed and closely spaced near the joints to strengthen the joints against the local failure and to achieve a monolithic behaviour.

7b. To prevent local stress failure cast-in-situ joints may be provided at critical locations. In this case, the

adjacent segments are placed about 100 to 150 mm. apart. The dowel bars (longitudinal bars) of the segments are welded. Concrete of high early strength cement is poured in the joint. Then, the other segments are arranged and prestressed. This method gives a nearly monolithic construction.

8. As far as possible the joints should be away from the critical locations.

9. In dry joints and cement slurry joints there is no tensile strength at the joint except the prestressing force. Therefore, for continuation of tensile stress across the joint, epoxy joint is preferred which gives the strength of the joint as much as that of concrete itself.

10. Thick cement slurry joints should be avoided as the hardened cement slurry spalls out and separates from the joint under pulsating loads resulting in a loss of prestress.

11. It has been observed that M.S. rods on the tension side of the section break due to large deflections and fatigue under pulsating loads. So, non-tensioned high strength reinforcement may be provided.

12. There is a discontinuation of tensile stress borne by M.S. bars after they break under pulsating loads. So, the cracks now behave like joints of some segmental construction.

Therefore the post-pulsating ultimate load capacity of all beams, monolithic and segmental, remain the same provided sections at links are identical. This was observed in all post-pulsating static load tests.

13. A judicious reduction factor should be taken for design of unbonded construction.

14. From the pulsating loadstests it is observed that the ratio of lower to upper load varied between 0.50 and 0.666. In case of long-span bridges, the ratio of dead load to total design load may be around 0.5 to 0.8. So, the present pulsating loads test can be said to be a model test of a long span bridge. For the design of highway bridges load factors considered are 1.5 for dead load and 2.5 for live load. In the present series of tests the lowest upper load considered was 54.6 percent of theoretical ultimate load. It was also found that the beam withstood approximately 0.3 million cycles between upper load and lower load as 54.6 percent and 36.4 percent of the ultimate load. In the design life of road bridges, the actual design load seldom passes over the bridge and in all cases there is a less probability of passing the live load more than 0.3 million times. So post tensioned segmental continuous beam construction can be recommended for highway bridges.

4.3 Scope of the Further Work:

This investigation is a very small effort and still vast scope of research exists in this field. Some of them are study of behaviour of continuous beams under pulsating loads with beam consisting of

i) segments of I and box - sections,

ii) bonded tendons,

iii) draped and curtailed tendons,

iv) varying cross-section along the span, and

v) epoxy joints.

APPENDIX - A

COMPUTATION OF LOSS OF PRESTRESS

Resultant prestressing force at jacking end $= P_t = 100.0$ kN

Net cross-sectional area of concrete $= A_c = 45900 \text{ mm}^2$

Total area of prestressing steel $= 78.5 \text{ mm}^2$

Modulus of elasticity of steel $= E_s = 2.036 \times 10^5 \text{ N/mm}^2$

Modulus of elasticity of concrete $= E_c = 5700 \sqrt{f_{ck}}$

$= 30589 \text{ N/mm}^2$

(Considering the lowest cube strength, 28.8 N/mm^2 to account maximum loss) .

Stress in tensioned steel $= f_t = \dfrac{100.0 \times 1000}{78.5}$

$= 1274 \text{ N/mm}^2$

Elastic strain in steel $= \epsilon_t = \dfrac{f_t}{E_s} = 6.26 \times 10^{-3}$

(i) Loss due to elastic shortening:-

Loss of strain $= \dfrac{P_t}{2\, E_c\, A_c} = 3.55 \times 10^{-5}$

(ii) Anchorage take-up:-

Assumed slip $= 2.5$ mm

Length of the beam $= 6300$ mm

Loss of strain due to anchorage take-up $= \dfrac{2.5}{6300}$

$= 3.97 \times 10^{-4}$

Other losses are neglected since prestressing was done after the concrete has matured and the test was conducted within few days of prestressing.

Thus

	Loss due to	Loss of strain	Percentage loss
1.	Elastic shortening	3.55×10^{-5}	0.57
2.	Anchorage take-up	3.98×10^{-4}	6.36
	Total loss	4.335×10^{-4}	6.93

Efficiency of prestress $= \eta = 1 - 0.0693$

≈ 0.93

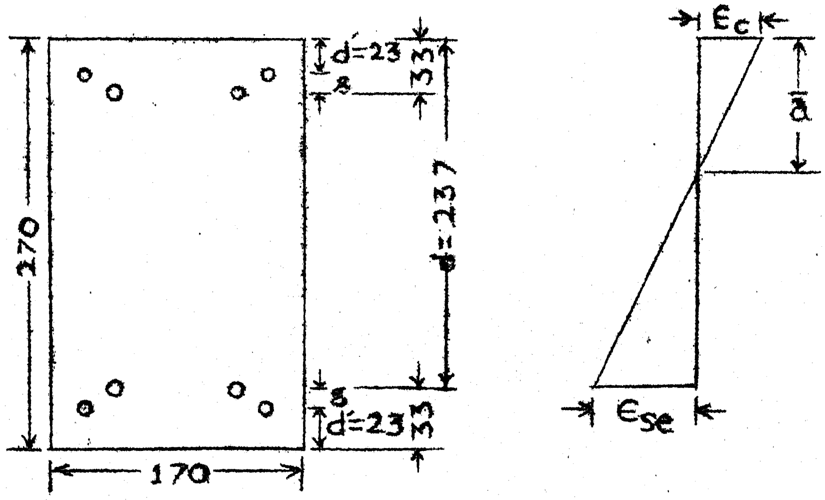
Effective prestressing strain $= \varepsilon_e = 5.82 \times 10^{-3}$

Effective prestressing force $= f_e = \eta \, f_t$

$= 1185$ N/mm^2.

APPENDIX - B

COMPUTATION OF ULTIMATE LOAD FOR BONDED SECTIONS



Assumed

Strain in H.T. wire at ultimate load = ϵ_s = 0.022

Compressive strain in concrete at ultimate load
= ϵ_c = 0.0035

Prestressing strain in steel = ϵ_e = 5.82×10^{-3}

Strain developed in steel = $\epsilon_{se} = \epsilon_s - \epsilon_e = 0.01618$

Depth of neutral axis can be computed from strain compatibility equation

$$\frac{\bar{a}}{d} = \frac{\epsilon_c}{\epsilon_c + \epsilon_{se}}$$

$$\bar{a} = 42.1 \text{ mm} \quad \text{(B.1)}$$

Depth of equivalent rectangular stress block

$$a = \frac{\bar{a}}{1.25} = 33.7 \text{ mm} \quad \text{(B.2)}$$

Compressive strain caused in compression side H.T. wire

$$= \frac{\bar{a} - d'}{\bar{a}} \cdot \varepsilon_c = 7.5 \times 10^{-4}$$

Therefore, net strain in steel $= 5.82 \times 10^{-3} - 7.50 \times 10^{-4}$

$$= 5.07 \times 10^{-3}$$

Corresponding stress from Fig. 2.1 $= f_{sc} = 1030.8\ N/mm^2$ (B.3)

Stress in H.T. wire on tension side corresponding to

$$0.022 \text{ strain} = f_{st} = 1607\ N/mm^2 \quad (B.4)$$

Compressive strain in M.S. bar = 0.00155

Corresponding stress $= f_c = 231.0\ N/mm^2$ (B.5)

Tensile strain in M.S. bar = 0.017

Corresponding stress $= f_t = 437.4\ N/mm^2$ (B.6)

$$A_{st} = A_{sc} = 39.2\ mm^2 \quad (B.7)$$

$$A_t = A_c = 56.55\ mm^2 \quad (B.8)$$

Strength of concrete for balanced section :-

$$A_{st}\ f_{st} + A_{sc}\ f_{se} + A_t\ f_t - A_c\ f_c = 0.67\ b.\ a\ f_{ck}$$

$$f_{ck} = 30.6\ N/mm^2$$

Moment capacity for balanced section:-

Due to H.T. wire alone

$$M_{rH.T.} = A_{st}\ f_{st}\ jd + A_{sc}\ f_{sc}\ (d' - \frac{a}{2})$$

$$= 14.493\ kN\text{-}m \quad (B.9)$$

Due to M.S. rods alone

$$M_{rM.S.} = A_t\ f_t(jd+s) - A_c\ f_c(d'-s - \frac{a}{2})$$

$$= 5.585\ kN\text{-}m \quad (B.10)$$

M_{p1} = Plastic moment capacity due to both H.T. and M.S. reinforcement

$= M_{rH.T.} + M_{rM.S.}$

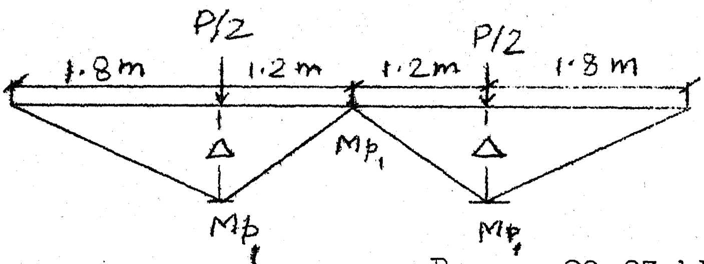
$= 20.078$ kN-m (B.11)

M_{p2} = Plastic moment capacity due to only H.T. wire

$= M_{rH.T.}$

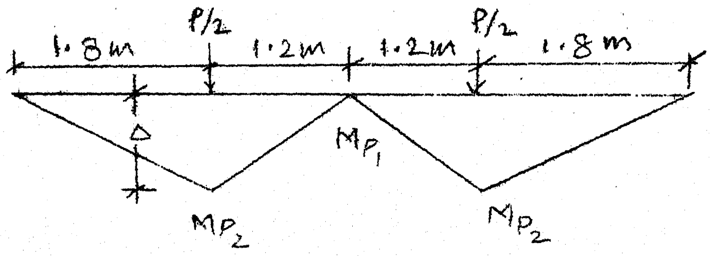
$= 14.493$ kN-m (B.12)

<u>For Monolithic Beam</u>



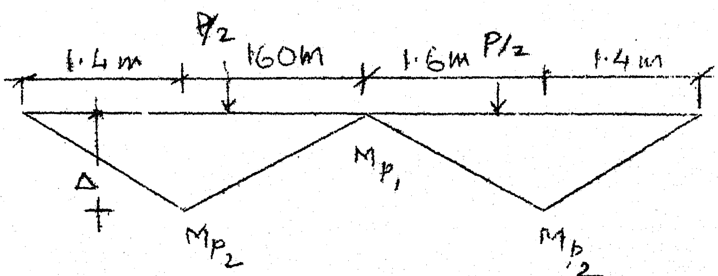
$P_{col} = 89.23$ kN

<u>For Beams with 5 Segments</u>



$P_{col} = 73.72$ kN

<u>For Beams with 7 Segments</u>



$P_{col} = 85.24$ kN

APPENDIX - C

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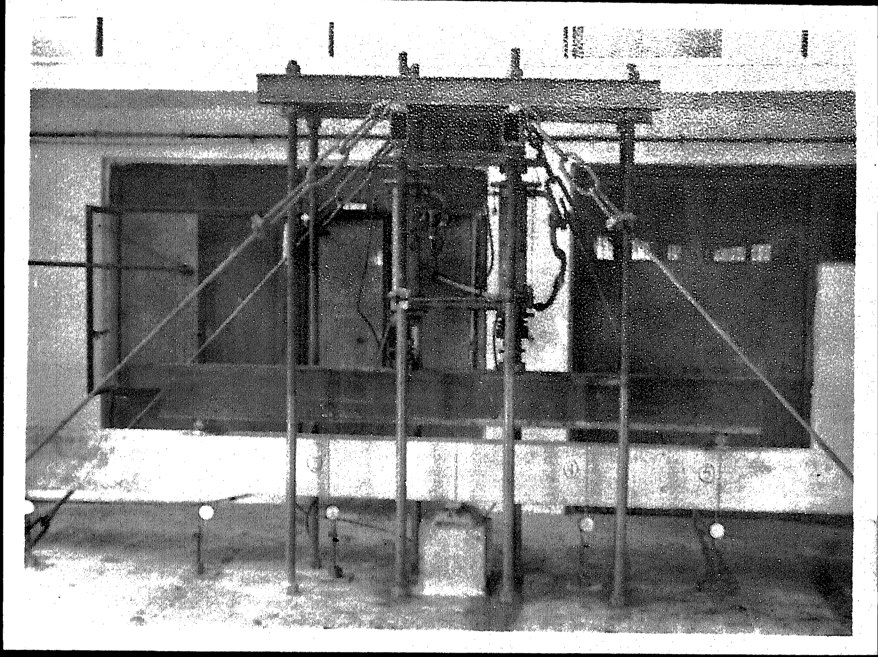
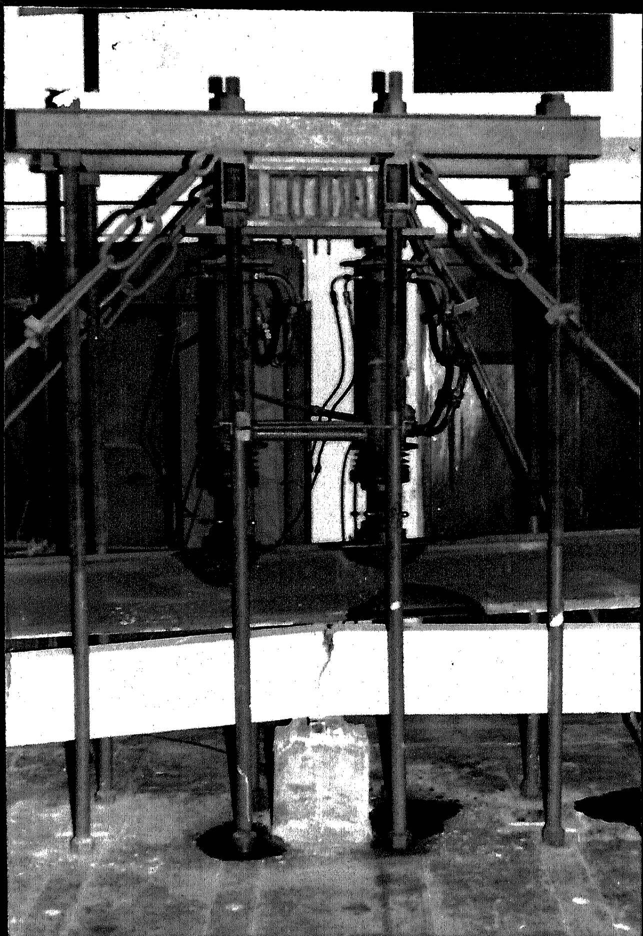
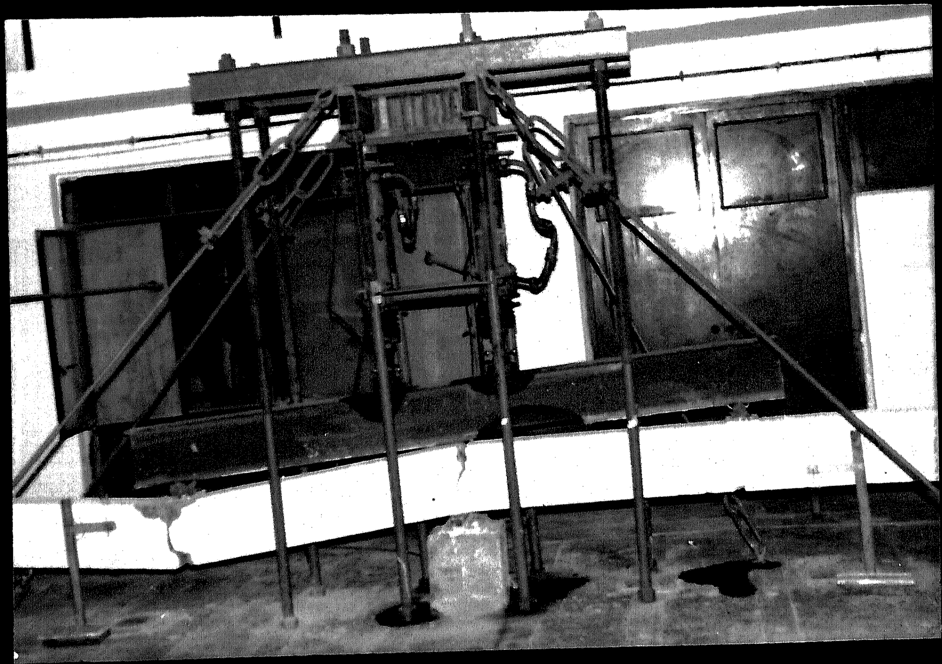


Beam Placed on the Loading Frame

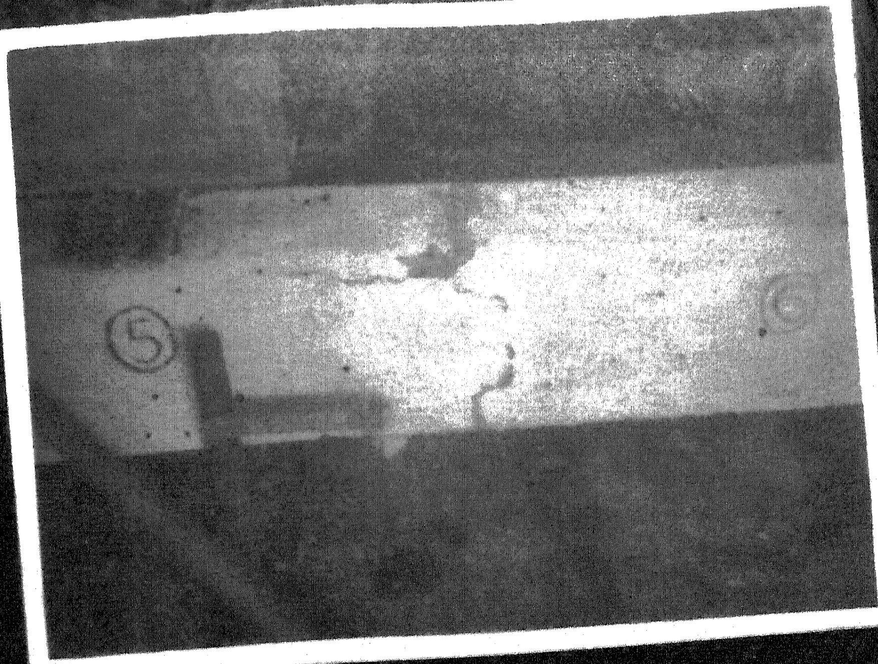


Central Crack in Beam 5B5S

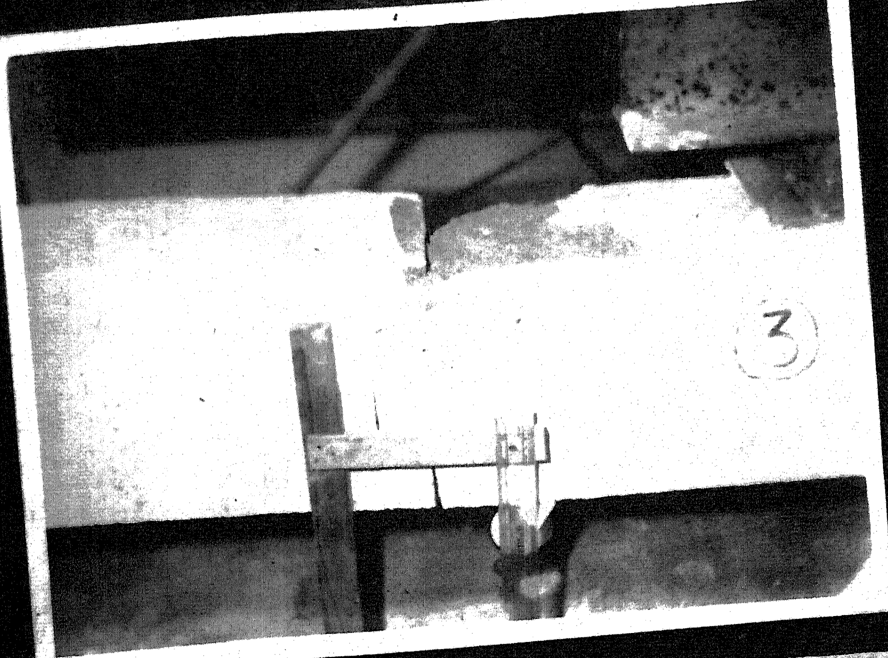


Beam 5B5S at Collapse

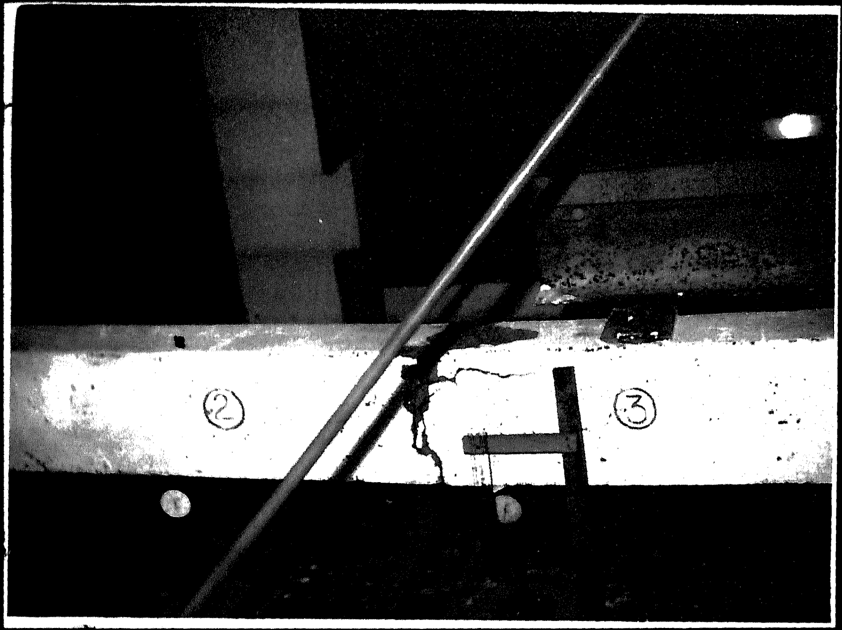
Tension Crack in Beam 7B7S



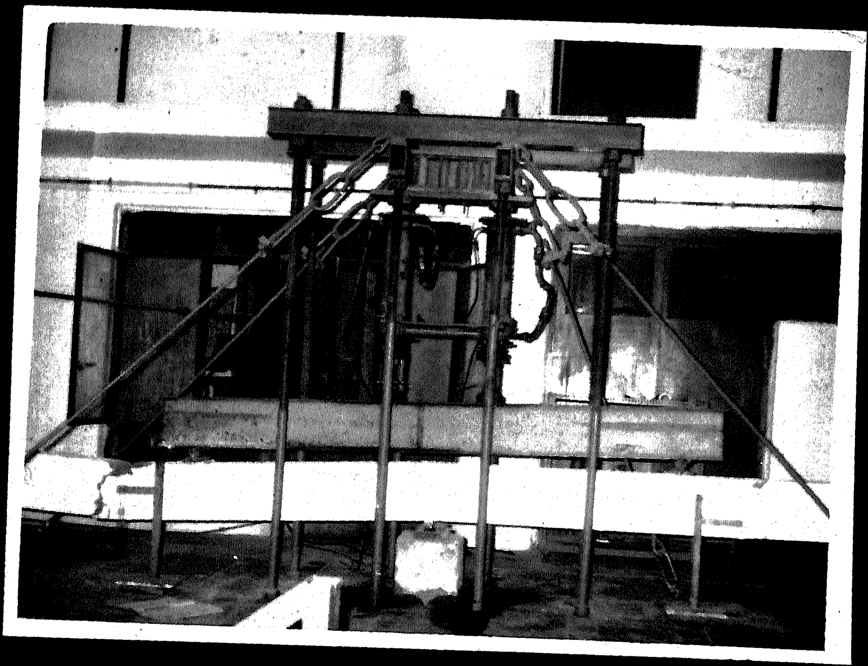
Failure at the Joint on the Right Span of
Beam 7B7S



Failure at the joint on the Left Span of
Beam 7B7S

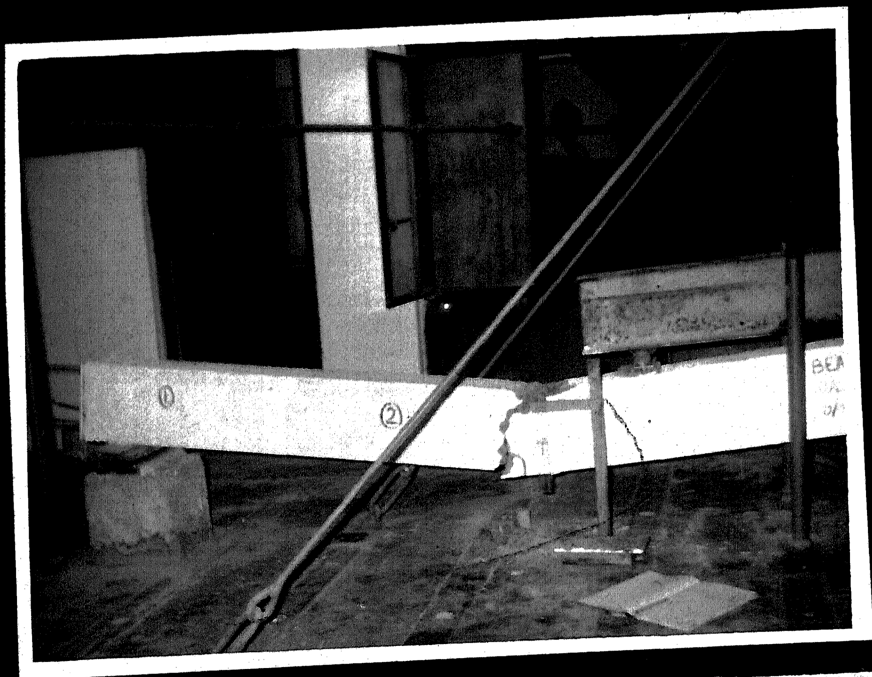


Failure at Joint of Beam 8B7S at the End of Pulsating Loads



Beam 9B9S at Collapse

Failure at Joint in Beam 1OB7S Before
Post Pulsating Load Test



Beam 1OB7S at Collapse